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The influence of floor structures on the seismic response of unreinforced masonry buildings from the late 19th and early 20th century in Zagreb

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Subject review

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The paper examines structural deficiencies of unreinforced masonry buildings of traditional Zagreb construction practices from Austro-Hungarian period, with an emphasis on the role of floor structures in the transfer and distribution of seismic actions. A historical overview of the development of such structures and building typologies is provided. Special attention is given to the insufficient interconnection of load-bearing elements and the uneven distribution of horizontal forces, which directly affect the seismic resistance of buildings. The paper analyses the properties of the materials used, typical floor plan layouts, and characteristic floor structure systems in Zagreb. Through three representative case studies, different structural system configurations are examined using two strengthening methods, based on which conclusions are drawn regarding the effectiveness of individual interventions.

Key words:

timber floor structures, masonry buildings, structural strengthening, Zagreb, Lower town, N2 method

Pregledni rad

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Utjecaj međukatnih konstrukcija na seizmički odziv nearmiranih zidanih zgrada iz razdoblja kasnog 19. i početka 20. stoljeća u Zagrebu

U radu su razmotreni konstrukcijski nedostaci nearmiranih zidanih građevina zagrebačke tradicionalne (austrougarske) gradnje, s naglaskom na ulozi međukatnih konstrukcija u prijenosu i raspodjeli potresnih djelovanja. Dan je povijesni pregled razvoja takvih konstrukcija i tipologija gradnje. Posebna pozornost posvećena je nedostatnoj međusobnoj povezanosti nosivih elemenata te neujednačenoj distribuciji horizontalnih sila, što izravno utječe na potresnu otpornost zgrada. U radu su analizirana svojstva ugrađenih materijala, tipične tlocrtne dispozicije te karakteristični sustavi međukatnih konstrukcija u Zagrebu. Kroz tri reprezentativne studije slučaja razmotrene su različite dispozicije konstrukcijskog sustava uz primjenu dviju metoda ojačanja, a na temelju čega su izvedeni zaključci o učinkovitosti pojedinih intervencija.

Ključne riječi:

drvene međukatne konstrukcije, zidane građevine, pojačanje konstrukcija, Zagreb, Donji grad, N2 metoda

1. Introduction

The city of Zagreb was first mentioned at its present location at the end of the 11th century. Over the centuries, it came under the rule of various rulers and state formations, while at the same time being exposed to a number of natural disasters; all these influences collectively guided its gradual urban and architectural development. In the 17th and 18th centuries, the city was significantly damaged by four major fires [1, 2], which prompted substantial changes in construction practices, leading to a transition from timber-based systems to brick materials. "During the 17th and 18th centuries, the first representative residential buildings of the nobility and bourgeoisie were constructed in Gradec" [2], while during the Habsburg Monarchy period, building types characteristic of Vienna were adopted.

With the change in construction methods, the residential building stock became more vulnerable to seismic actions, primarily due to the use of massive materials with high volumetric weight, which generate significant inertial forces under seismic excitation. Earthquakes of high intensity were recorded in Zagreb in 1880, 1883, 1901, 1905, and 2020 [3, 4]. The earthquake of magnitude M_w = 6.3 that struck Zagreb in 1880, known as the Great Zagreb Earthquake, caused extensive damage to the building stock, with more than 1,700 buildings affected. Architects and builders from various parts of the Austro-Hungarian Monarchy participated in the reconstruction, significantly influencing the city's further development.

During the reconstruction, major urban and architectural transformations took place, including the reshaping of many existing streets and buildings. The reconstruction process lasted for several years, with some buildings completed only in the early 20th century. This extended timeframe resulted in a layered architectural expression, in which various styles characteristic of the period can be recognized. After the earthquake, a shift toward a more uniform and standardized design approach within historicism is observed, while in later stages elements of Secession and modern architecture gradually appear. The residential building stock of the protected historic urban ensemble of the City of Zagreb predominantly dates from the late 19th and early 20th centuries [5]. Buildings from this period were systematically organized into closed urban blocks (Figure 1), defined by street fronts and inner courtyards. Such block construction resulted in continuous rows of buildings with adjoining walls and a clearly defined urban matrix. Load-bearing walls are generally arranged predominantly in the direction parallel to the street, while in the direction perpendicular to the street insufficient percentage/number of walls is observed. In the study by Stepinac et al. [6], based on a representative sample of 103 analysed buildings, it was determined that the wall percentage of 5 % in plan is not satisfied in 12 % of buildings in the x-direction (parallel to the street) and in 16 % in the y-direction (perpendicular to the street). Under a stricter criterion, i.e., a threshold value of 10 % of wall area in plan, approximately 75 % of buildings do not meet the requirement, regardless of direction [6]. The study also provides a detailed analysis of parameters such as wall slenderness, plan asymmetry, aspect ratio, projection length, compactness, and the relative position of the center of mass and center of stiffness.

The primary load-bearing system of these buildings consists of massive load-bearing brick walls, with shallow "Prussian" vaults constructed above the basement or semi-basement levels, while in the remaining floor zones timber joist systems are used, with infill (rubble or debris) placed between the joists to provide acoustic and thermal insulation. The roof structure is typically made of timber, most commonly in a pitched (gable) form, with traditional clay tile covering, where the loads are transferred to the longitudinal load-bearing walls (Figure 2).

By analysing buildings in the historic city center of Zagreb, it is evident that structures aged between 100 and 150 years predominate, whereas in the design of new buildings, bridges, and other engineering structures of conventional size or ordinary importance, designers typically assume a service life of 50 years [7, 8]. In addition to the fact that their service life has long been exceeded, historic buildings do not satisfy fundamental principles of construction in seismic regions, one of which is the presence of a sufficiently rigid diaphragm at floor level, essential for the proper distribution of horizontal inertial forces to vertical load-bearing elements. A lack of structural connectivity, besides posing issues of global stability, also leads to pronounced out-of-plane failure mechanisms. Out-of-plane failures represent one of the most common and most dangerous forms of mechanical instability in masonry buildings during earthquakes [9]. The construction typology of historic buildings in Zagreb directly influences the out-of-plane failure mechanism of walls due to the lack of stiffening walls perpendicular to the load-bearing street-facing walls. Over

Figure 1. Typical Lower Town urban block in Zagreb

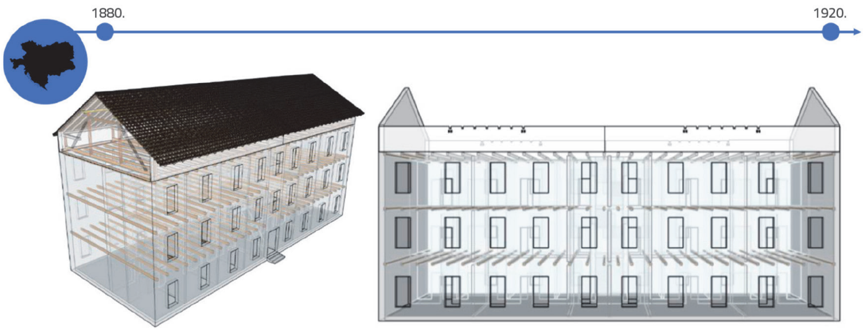

Figure 2. A typical residential building in the historic city center of Zagreb

the service life of these buildings, which often exceeds 150 years, many of the stiffening walls perpendicular to the street have been removed through various unauthorized and structurally unacceptable alterations. The first seismic code, introduced in 1964, defined in its initial provisions the maximum allowable spacing of stiffening walls depending on the intensity of seismic action, i.e., the seismic zone. This fact clearly demonstrates the importance of preserving these walls within the structural system. Residential buildings in Zagreb were affected by a moderate-intensity earthquake in 2020, and is hard to compare the damage caused by earthquakes of magnitudes 7.7 and 7.6 M_w that struck Turkey [10]. In the historic urban ensemble of the City of Zagreb, ground-floor spaces used for retail and hospitality purposes are typically modified by removing primary load-bearing walls, which, during high-intensity earthquakes, leads precisely to the activation of the soft-storey mechanism.

2. Historic unreinforced masonry buildings in Zagreb

2.1. Materials used in construction

Historic buildings in the core of Zagreb were predominantly constructed using brick masonry combined with lime mortar as a binding material. The strength ratios of materials used in masonry structures are highly non-uniform [11]. Since these materials already exhibited varying mechanical properties at the time of construction, and have undergone long-term degradation, it is essential to determine the current mechanical properties when developing advanced nonlinear numerical models.

2.1.1. Brick and lime mortar

Brick elements are a construction material produced industrially, with more uniform mechanical properties and dimensions compared to natural materials. Until 1932, the so-called Austrian brick with dimensions of 14 cm × 29 cm × 6.5 cm was used, while after 1932 there was a transition to bricks of the so-called standard format, measuring 12 cm × 25 cm × 6.5 cm. In historic buildings, brick elements exhibit significant material degradation, particularly in zones exposed to soil contact and moisture. Since modern waterproofing systems were not implemented, this directly contributes to a reduction in the mechanical properties of the material. Alongside brick, the primary load-bearing component in masonry structures is lime mortar. Lime has been traditionally used as a construction material for more than three thousand years [12]. It is produced by heating limestone to convert calcium carbonate into calcium oxide (quicklime), which is then slaked with water [13] followed by the addition of sand and water to obtain lime mortar. This composite material, consisting of brick and lime mortar, exhibits highly complex behavior and mechanical properties. Two key parameters that must be determined for such masonry are the modulus of elasticity and the shear strength of the material. Today, two established methods are commonly used for testing. The simpler, less expensive but also less accurate method involves measuring the force at which failure occurs in the mortar joint without measuring vertical load. The second method, significantly more accurate but also more complex and time-consuming, involves the use of flat-jack testing, which enables determination of the actual vertical stress within the masonry. Additionally, new approaches for evaluating mechanical properties are being explored, such as measuring the velocity of sound waves passing through the material, as investigated by Ortega et al. in [14].

The calculation parameter that would best represent the properties of masonry is its tensile strength. However, due to the complexity of testing and the highly destructive nature of tensile tests themselves, it is almost impossible to perform and financially justify such testing on buildings that have not suffered significant damage, i.e., buildings with a damage grade lower than IV according to the EMS-98 classification. Such testing is highly complex, expensive, and time-consuming, making it impractical to carry out on a large number of buildings. Consequently, the results would likely exhibit considerable variability and would not be sufficiently representative of the entire building stock.

Within the ARES project, numerous tests were conducted on the shear strength and modulus of elasticity of masonry in Zagreb and its surroundings. The results were published in the doctoral dissertation of Luka Lulić, as well as in a scientific paper by Stepinac et al. [15, 16]. The obtained results are highly heterogeneous. It is important to note that, for certain buildings, the calculated values (Figure 3) are below the recommended modulus of elasticity of E = 1500 MPa, as proposed by the new generation of Eurocodes and the Italian national standards (NTC 2018 and NTC Circolare 2019).

Nine buildings exhibit a modulus of elasticity within the range of 500 MPa to 1100 MPa, while four buildings record values exceeding 2000 MPa. This clearly demonstrates why it is not appropriate to apply advanced nonlinear analyses without prior testing of the mechanical properties of materials when designing structural strengthening interventions.

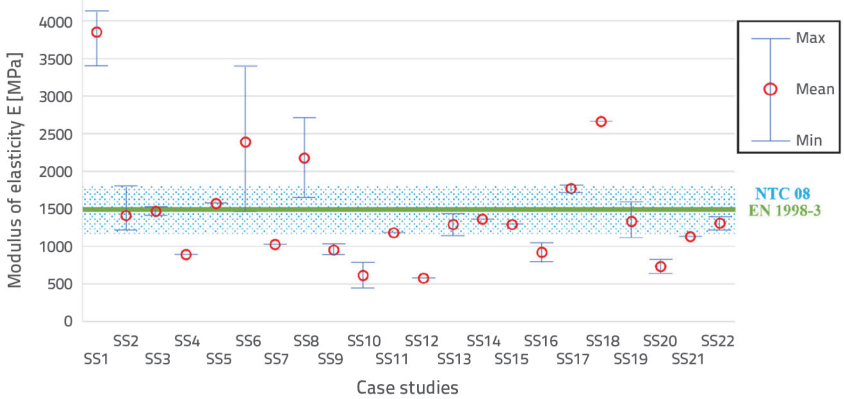

Figure 3. Results of modulus of elasticity testing in comparison with EN and NTC standards [15]

2.1.2. Timber

The botanical species of wood most commonly found in buildings in Zagreb are fir and spruce, while oak was typically reserved for buildings of special importance or for zones with high stress concentrations. Wood as a material is characterized by variations in mechanical properties depending on its moisture content, and excessive moisture can lead to mechanical degradation.

Floor structures generally do not exhibit significant degradation due to biological agents, unlike roof structures shown in Figure 4. The degradation of timber due to rot caused by the presence of moisture in floor structures occurs primarily in the beam-end support zones, specifically in the portions of the timber joists embedded within the wall where ventilation is not provided. This is expected considering the principles of building physics and the typical detailing of timber joist bearings in masonry walls. Timber joints within floor systems are "preserved" by layers of rubble and sand, and damage typically occurs in areas where these layers have been removed or where new timber elements with excessive moisture have been introduced. During the rehabilitation and strengthening of floor structures, it is essential in the initial phase to review archival documentation, conduct a preliminary visual inspection, perform geometric measurements, and subsequently carry out a detailed assessment of interventions and connections within the structure [17].

When designing rehabilitation and strengthening measures for existing timber structures, a significant issue is the lack of relevant normative documents governing such processes. Across Europe, the need to define standards for the assessment of existing timber structures has been recognized, and several countries have already addressed this issue [18]. Botanical species, structural types, element positions within structures, and similar factors directly influence structural behaviour. In Croatia, available data are typically linked to individual scientific or professional studies, with three key sources often used as references. The Technical Museum in Zagreb, constructed in 1949, was analysed in a study by Stepinac et al. [19] showing that elements in exposed zones with higher moisture content correspond to strength class C16, while elements in elevated zones above ground level can be classified as C24, and in some cases even C27. In the study by Perković et al. [20] the timber roof structure of the Croatian State Archives was analysed, concluding that the predominant strength class is C24. It should be noted that both of these buildings are of high importance, and therefore the material quality may be superior compared to typical residential buildings. For laminated timber roof structures, mechanical properties were investigated on an industrial facility in Kutina, constructed around 1980 [21]. Test specimens taken from glued laminated beams were subjected to destructive laboratory testing, yielding a characteristic bending strength of 22 N/mm², placing them below the GL24h strength class, which is today considered a standard class.

Figure 4. Timber structure damaged by biological agents

Based on the analyses and tests carried out in the aforementioned studies, it can be concluded that, for existing timber structures, it is essential to consider the effects of moisture and material degradation, along with a reduction of the effective cross-sectional area when calculating geometric properties.

2.1.3. Steel

In historic residential buildings, steel is used to a very limited and localized extent, primarily in positions where high loads are concentrated and where other materials with lower strength cannot meet the required mechanical properties. Steel beams are frequently installed beneath stair flights as stringers, beneath partition walls at floor level, in balcony zones, in conjunction with Prussian vaults in ceilings above basements or semi-basements, and in geometrically more complex floor plan

Table 1. Geometric characteristics of the historical NPI 200 and modern IPE 200 steel profiles

Profile	**Height** [mm]	**Width** [mm]	**Flange thickness** [mm]	**Mass** [kg/m]
IPE 200	200	100	5.6	18.4
NPI 200	200	90	7.8	22.4

configurations that cannot be spanned using timber joists, as shown in Figure 5.

Figure 5. Steel floor joists in a typical Lower Town corner building in the bay window zone, and steel beams beneath partition walls

Steel profiles used historically had different geometries compared to those used today. Historical profiles are characterized by smaller widths, thicker flanges, and greater mass per unit length. They are generally more massive and less optimized than modern profiles. Table 1 presents a comparison of geometric characteristics between a historical NPI 200 profile and a modern IPE 200 profile.

2.2. Typology of historic buildings in Zagreb

Masonry historic buildings represent the dominant structural system in the Lower Town of Zagreb. Following the 2020 earthquake, Stepinac et al. [22] analysed the types of structural systems. Figure 6 shows darkerk brown tones in the areas of the Lower and Upper Town, indicating a high concentration of masonry buildings in the historic city core of Zagreb. In the study by Moretić et al. [5] three urban blocks were analysed, including parameters such as year of construction, number of storeys, and type of floor structure. Approximately 50 % of buildings date from the period between 1861 and 1920, while the next most represented period is from 1921 to 1945, accounting for slightly more than 10 %. Among these buildings, four-storey structures are the most common, representing around 40 %, while five-storey buildings account for about 15 %. The most prevalent type of floor structure is flexible timber floor systems, with a share of approximately 60 %, whereas reinforced concrete rigid diaphragms account for less than 5 % within the analysed three urban blocks.

The development of Zagreb was strongly influenced by the Habsburg Monarchy, under whose rule it remained for centuries. The historic urban ensemble was developed following the model of Vienna, resulting in the formation of a series of urban blocks. The block layout significantly influenced the floor plan configurations used for residential buildings. While block construction allows for efficient land use, it also presents several drawbacks in terms of flexibility in floor plan design and natural lighting of residential spaces. These constraints led to highly articulated and irregular floor plans, which today pose significant challenges from a structural engineering perspective when strengthening load-bearing systems.

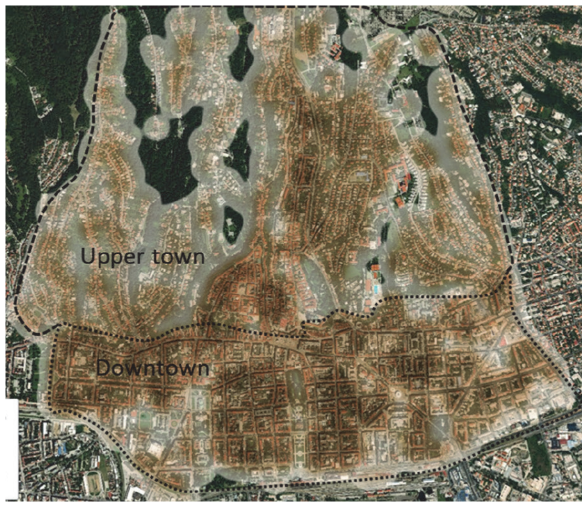

Figure 6. Prevalence of masonry structures in the historic city center of Zagreb [22]

Based on analyses and inspections of a large number of buildings following the 2020 earthquake, three characteristic floor plan types with significant prevalence in the historic core were identified. Figure 7 presents the characteristic ground-floor layout of Type 1, from which it can be observed that the load-bearing walls are arranged parallel to the street in four wall lines.

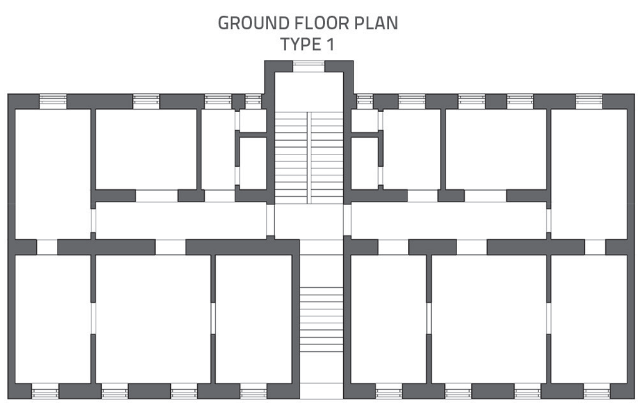

Figure 7. Ground floor plan layout – characteristic Type 1

Perpendicular to the main load-bearing walls are very thin partition stiffening walls, whose function is to prevent the out-of-plane instability of the street-facing walls. As a rule, these walls are bonded into the masonry together with the street-facing walls; however, their thickness is most often insufficient to resist seismic forces. Following the 2020 earthquake, significant diagonal and X-shaped cracks were observed in these walls.

In the courtyard section, a prominently projecting staircase is visible, with the extent of its projection varying among different buildings. The transverse walls (the façade walls and the staircase walls perpendicular to the street) are not subjected to vertical loading, since the floor joists are oriented perpendicular to the load-bearing street-facing walls. This further reduces the seismic resistance in the transverse direction. This type of floor plan does not ensure bidirectional resistance and stiffness, which represents one of its main deficiencies, along with the insufficient stiffness of the floor diaphragm.

Figure 8 shows the characteristic ground floor layout of Type 2, which features two protruding volumes in the courtyard zone, providing improved natural lighting for courtyard-facing rooms on deeper building plots. In this way, a more architecturally favorable layout is achieved; however, it clearly does not satisfy the requirements of structural uniformity, symmetry, and plan regularity.

Compared to Type 1, the load-bearing walls in this layout are more evenly distributed, but the issue of insufficient floor diaphragm stiffness at the storey level still remains.

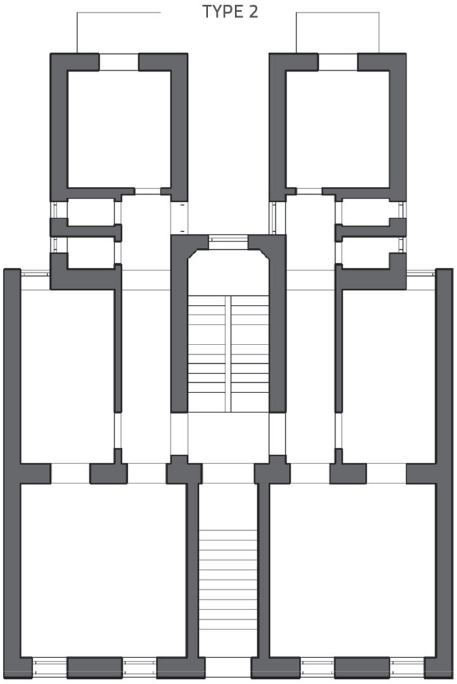

Figure 8. Ground floor plan layout – characteristic Type 2

The floor plan layout shown in Figure 9 represents buildings located at the corners of urban blocks, commonly referred to as "corner buildings." This type of structure is characterized by the most unfavorable wall distribution, geometric configuration, and compliance with fundamental principles of seismic design. In the analysed existing buildings of this type, torsional behavior is very frequently observed in the initial modes. Strengthening measures for such structures typically require extensive construction interventions, particularly the addition of new walls to adequately resist seismic forces.

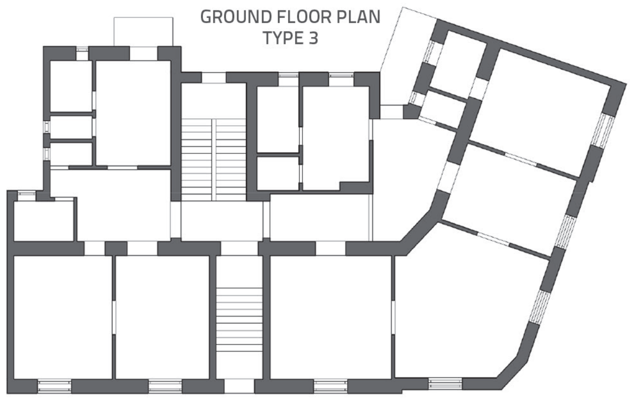

Figure 9. Ground floor plan layout – characteristic Type 3

3. Floor structures of historic buildings in Zagreb

Floor structures, in addition to transferring permanent and live vertical loads, play a crucial role in seismic regions by ensuring structural integrity and distributing seismic forces to the walls. At the end of the 19th and the beginning of the 20th century, timber floor structures were most commonly used, typically spanning up to 6 meters, with a minimum clear span of 3.5 meters and a usual thickness ranging from 30 to 55 cm, depending on the type and construction method [11]. Such floor systems act as flexible diaphragms that are not adequately connected to the main load-bearing walls, do not provide sufficient stiffness, and distribute inertial forces to the walls independently of their stiffness, based only on their share in the plan area. In this type of floor structure, local impacts of joints on the supporting walls may occur, and due to insufficient connectivity and lack of anchorage, local wall failure can develop at support zones. When developing numerical models of masonry buildings, the fundamental prerequisite for a valid global model is ensuring sufficient structural connectivity at the floor levels. If this condition is not met, analyses such as in-plane bending or shear of masonry do not realistically represent structural behavior, as there is a high probability of activating local failure mechanisms of masonry elements. In the study by Jajčević et al. [9] local failure mechanisms are described in detail, including calculation methods based on the principle of virtual work, and typical examples are identified, one of which is related to façade wall failure caused by insufficient connection between floor structures and load-bearing walls. This paper does not address local out-of-plane failure mechanisms; instead, the case study focuses on strengthening interventions aimed at ensuring global structural integrity.

3.1. Types of floor structures

3.1.1. Floor structures with traditional timber joists

Floor structures composed of timber represent the dominant structural system in historic buildings in Zagreb. In cases where the floor depth was not limited, a system with timber beams without a recessed infill layer was used. The total thickness of this system is approximately 39 cm. Beams are typically spaced up to 90 cm apart, while their cross-sectional dimensions vary depending on the span. A layer of planks was placed over the timber joists, followed by a layer of sand or rubble, into which bedding pads were set for the installation of the upper wooden decking and, ultimately, the floor finish. On the underside, this system consisted of a layer of boards nailed to the beams, onto which reed mats and a plaster layer were applied. The wooden decking was not additionally connected to the walls, and the timber beams were designed as simply supported elements. A schematic representation of this system is shown in Figure 10.

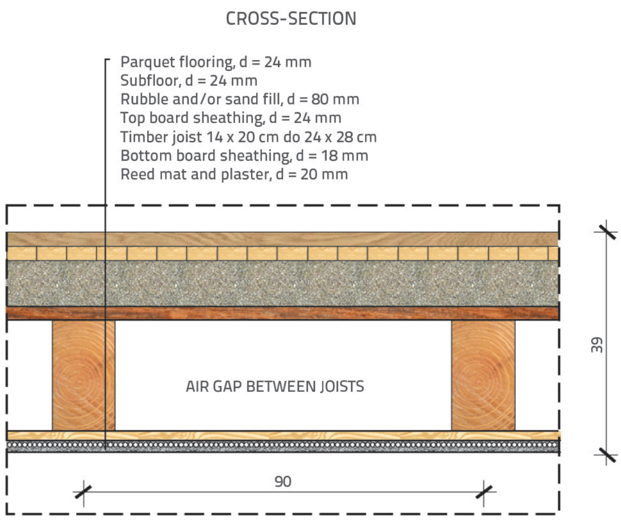

Figure 10. Cross-section of a floor structure with traditional timber beams without recessed infill

A very similar system with timber beams (Figure 11) represents a second type, differing only in the recessed upper wooden decking. Timber battens or ledgers were installed along the sides of the beams, supporting the upper boarding. By introducing recessed decking, the insulating layer of rubble or sand was partially placed between the beams, resulting in a reduced overall thickness of the floor structure. When surveying existing structures, the overall thickness of the floor system can be used to make preliminary assumptions about the layer composition, based on the described construction types.

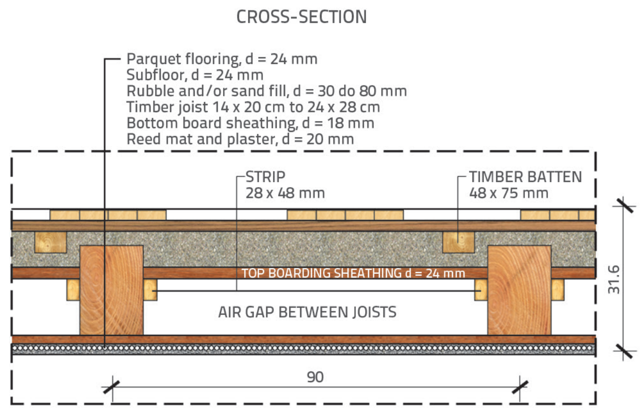

Figure 11. Cross-section of a floor structure with traditional timber joists with recessed infill

In previous sections, it was stated that timber beams are generally in satisfactory condition, provided that the positions of wet areas within apartments have not been altered and that no elements with excessive moisture have been introduced into the structure, which could lead to the development of biological agents. One of the reasons for this condition is the ventilation of the floor systems. On façade walls, at floor structure levels, perforated gypsum tiles are often installed at intervals of 90 to 100 cm to ensure a favorable microclimate within the floor structure. This detail must be replicated during rehabilitation or strengthening of the floor structure, as it ensures durability and represents significant conservation value (Figure 12).

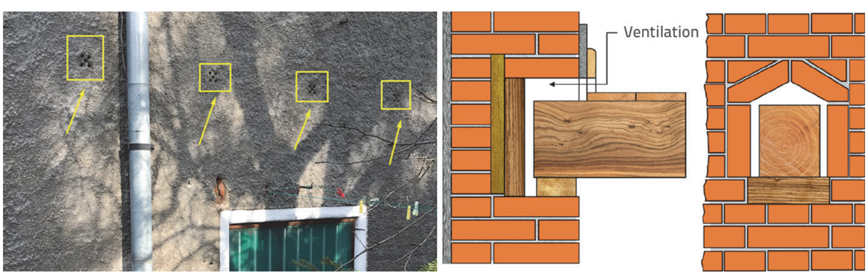

Figure 12. Perforated gypsum elements at the floor structure level on the façade and an example of ventilation

3.1.2. Floor structures with double timber joists

The system with double timber beams, shown in Figure 13, was used in public buildings such as healthcare facilities and schools in order to provide separate load-bearing systems for the floor and ceiling. This configuration helped prevent the transmission of sound and vibrations between storeys. The system was also applied for larger spans. Timber beams were arranged in two planes: the main floor beam carried permanent and live loads, while the ceiling beams were offset from the floor beams to allow for separation and movement, carrying only the load from ceiling layers. In all timber systems, the floor layers above the upper boarding and the ceiling layers on the underside do not differ significantly. The main disadvantage of this floor system is its considerable thickness, which is approximately 51 cm.

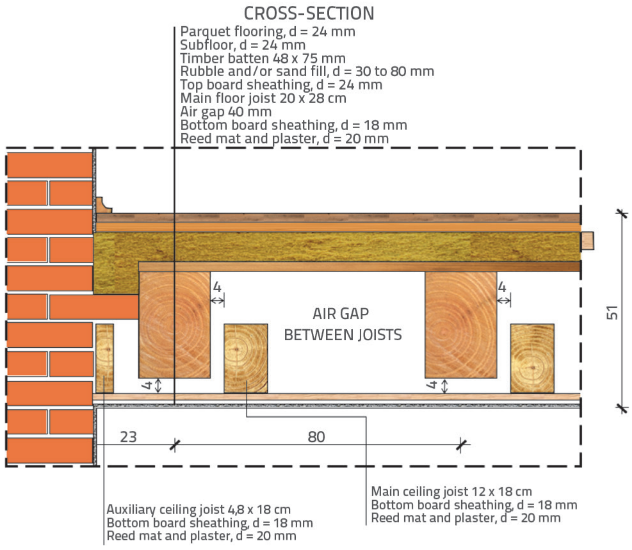

Figure 13. Cross-section of a floor structure with double timber joists

3.1.3. Floor structures with three-side hewn beams

The floor structure with three-side hewn beams, shown in Figure 14, represents a system that contributes to better connectivity of the main load-bearing wall elements provides greater stiffness compared to traditional timber joist systems spaced at 90 to 100 cm. This system is most commonly found in buildings dating from the mid-19th century, which are often individually protected cultural heritage structures.

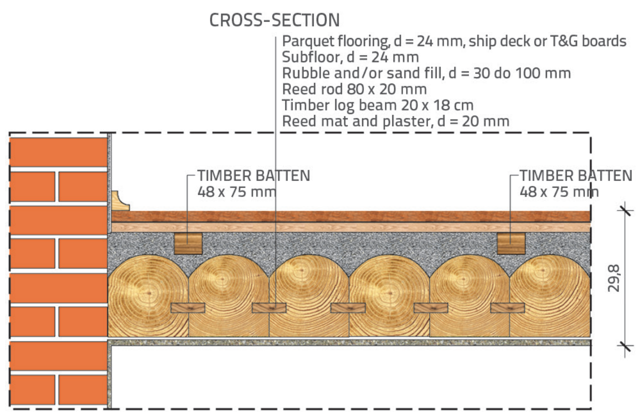

Figure 14. Cross-section of a floor structure with three-side hewn beams

It was also historically used in buildings with storage functions, where higher live loads occur. In buildings dating from the late 19th century, alongside conventional timber systems in other storeys, three-side hewn beams were typically used for floor structures beneath attic spaces. In these cases, brick tiles ("tavelice"), approximately 4 cm thick, were placed within the infill layer to provide fire resistance and prevent the spread of fire from the attic zone. Attic spaces historically served as storage areas and therefore posed an increased risk of fire ignition and propagation to other parts of the building. It is worth noting that "in 1868, Viennese fire protection regulations were introduced, prescribing this method of fire protection" [11].

In this system, the timber beams are hewn on three sides, while the upper side retains the original rounded shape of the log (Figure 15). The beams typically have a cross-section of 20 cm × 18 cm and are placed tightly next to each other. It is also evident that they are interconnected using wooden dowels to achieve better structural connectivity and stiffness. A fill layer of varying thickness, depending on the position, was placed on top of the beams. Within this layer, bedding pads and wooden decking were installed if parquet flooring was used, or brick tiles were laid directly onto the fill layer. In addition to greater stiffness and higher load-bearing capacity, this type of floor structure is characterized by a relatively small structural depth. However, it also requires significantly more timber, which consequently results in substantially higher construction costs.

Figure 15. Floor structure made of three-side hewn beams in a residential building and a monumental structure

3.1.4. Floor structures with timber joists and steel profiles

Steel profiles in the floor structures of historic buildings are used in zones where the use of timber elements, due to their mechanical properties and material limitations, would not be economical. In floor plan configurations—most commonly in corner buildings that lack regular square or rectangular geometry—steel beams are introduced in areas with high load concentrations or large spans. Timber beams are then placed on these steel beams, carrying loads in the shorter span direction. By organizing the structural system in this way, it consists of primary steel beams and secondary timber beams, allowing optimal use of the mechanical properties of each material. At the beginning of the 20th century, floor structures in more representative public buildings with larger spans commonly employed steel sections combined with a 10 cm thick reinforced concrete slab cast between the steel members. A similar arrangement was also used in cases where secondary timber beams were positioned perpendicular to the main girders. In these structural systems, the depth of the steel sections exceeded the thickness of the reinforced concrete slab.

The space above the slab and up to the top flange of the steel section served as a layer for thermal and acoustic insulation and was typically filled with rubble or construction debris. Another typical application of steel beams is beneath partition brick walls when these walls are not continuous, as shown in Figure 16. Brick elements have significant self-weight, and timber joists are often unable to meet deflection requirements over medium or larger spans; therefore, steel girders provide an optimal solution in such cases. During visual inspection of a building, the positions of steel elements cannot usually be identified without investigative works, as they are embedded within the floor layers. The floor and ceiling layers of this type of structure are otherwise identical to those of traditional timber joist floor systems without recessed infill.

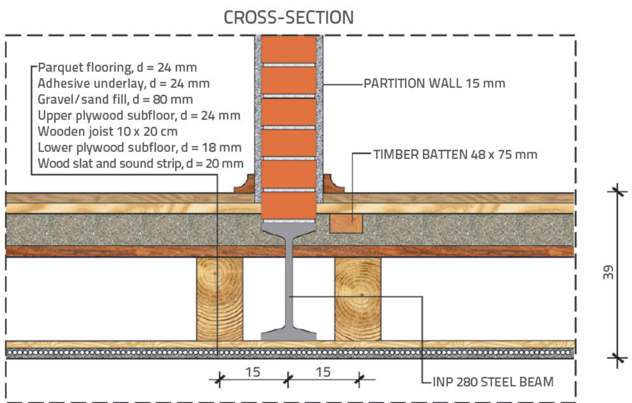

Figure 16. Cross-section of a floor structure with timber joists and steel profiles

3.1.5. Floor structures with shallow masonry vaults and steel beams

The use of brick vaults found its application in zones above basements or open ground floors, in order to protect the structure from moisture penetration from lower levels, as well as from the spread of fire toward residential spaces (Figure 17).

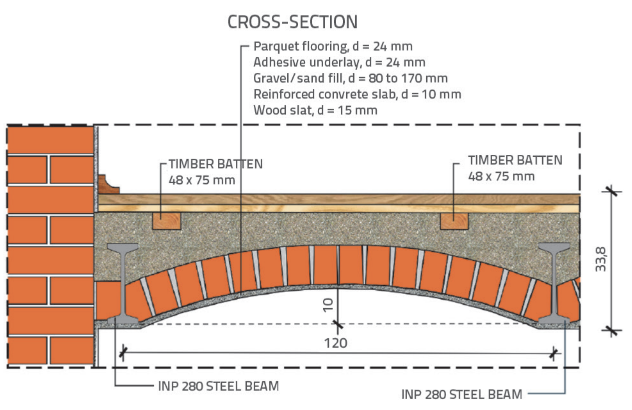

Figure 17. Cross-section of a floor structure with shallow masonry vaults and steel beams

For smaller spans, inclined semicircular vaults were used, but more commonly the so-called "Prussian" vault was applied due to its low structural height and easier adaptation to design requirements. Steel girders are installed at each end of the vault, while the rise of the vault is approximately one-tenth of its span. The vaults are constructed in a slightly curved form, and this system is used, in addition to the previously mentioned applications, in corridor and staircase areas, for example, in the construction of landings, where fire resistance and service load requirements are particularly demanding.

3.2. Strengthening methods for timber floor structures

When carrying out the rehabilitation and strengthening of timber floor structures, it is first necessary to assess the existing condition of the structure. Numerous non-destructive methods and models can be used to evaluate the condition and the mechanical and physical properties of timber elements and details, but all methods have certain limitations [23]. During structural rehabilitation, floor structures are typically opened only locally at several points within the building, making it often difficult to carry out thorough investigative works and testing. Strengthening of floor structures can be classified into several groups. In seismic retrofitting, some type of compression layer (topping slab) is commonly applied to stiffen the structure and connect it to the walls, and such interventions can be classified as the first group. In addition to constructing a topping slab, i.e., a sufficiently rigid floor diaphragm, it is essential to provide an effective connection between the diaphragm and the load-bearing walls. The primary objective is not to further strengthen the floor structure to carry increased vertical loads, but rather to create a sufficiently rigid diaphragm and ensure its connection to all load-bearing walls, thereby achieving greater overall resistance to seismic action. The second group includes measures aimed at improving the connection between timber beams and walls, along with localized repair of beams where moisture damage or biological deterioration is observed. The third group comprises interventions focused on strengthening the beams themselves in cases of excessive deflection or linear damage, typically by adding new timber or steel elements.

3.2.1. Construction of a compression layer (topping slab)

Strengthening timber floor structures by adding a timber–concrete composites (Figure 18) represents one of the most rational interventions, as it maximizes the high tensile strength of timber in the tension zone of the cross-section and the high compressive strength of concrete in the compression zone. These two materials are connected into a composite system using high-strength, ductile connectors (typically screws), which transfer stresses between elements through shear. When selecting connectors, preference should be given to those with low slip capacity in order to minimize relative movement. The stiffness and strength of the system must be quantified and considered in the design process, with particular attention paid to the ductility of the connection itself [24]. In practice, this type of composite beam is analysed using the γ-method, which applies to the elastic range of behavior. Daanoune et al. [25] investigated the nonlinear behavior of timber–concrete composite beams through numerical modeling in advanced software packages. The thickness of the concrete slab is usually limited to about 8 cm, as greater thickness would significantly increase the self-weight of the structure [26]. Increasing the thickness of the topping slab beyond 8 cm does not contribute to an increase in the resistance of the composite cross-section, as the joists are relatively shallow. For this reason, as well, the use of thicker slabs is considered uneconomical and structurally unjustified.

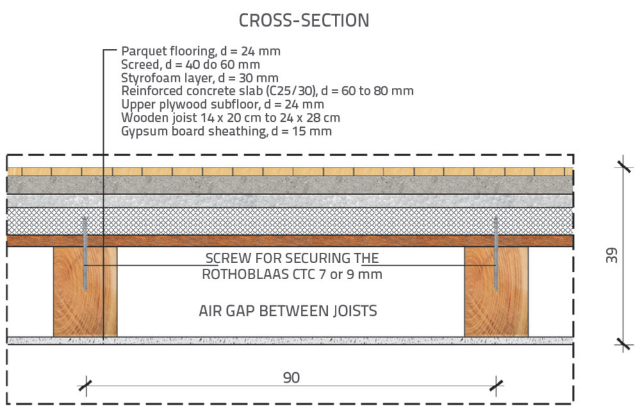

Figure 18. Strengthening of a floor structure with traditional timber joists without recessed infill using a reinforced concrete compression slab

The reinforced concrete compression slab is most commonly connected to timber beams using screws arranged in one or two rows, depending on the width of the timber element. These connectors are typically installed at angles of 30° or 45°, either as single screws at each position or as pairs arranged in an X-configuration to achieve higher load-bearing capacity. In support zones, where shear stresses are higher, the screws are placed at closer spacing, while farther from the supports they are spaced more widely for reasons of economy and efficiency. After the installation of the connectors, a reinforcement mesh is placed within the compression zone and additionally anchored into the walls. By implementing this strengthening method, the structure is partially unloaded through the removal of rubble infill. After completion, deflections are reduced compared to the existing condition, and the load-bearing capacity under vertical actions is increased.

In addition to concrete as the material used for the compression layer, timber-based materials can also be applied, including CLT panels, glued laminated panels, veneer panels, and LVL panels. Mirra, in his doctoral dissertation and scientific publications, provides a detailed overview of strengthening methods using different types of panels [27]. In the historic urban ensemble of Zagreb, structural rehabilitation often involves systems where the joists are stiffened from the underside by removing the ceiling layer, without interventions in the remaining floor layers. One such example is shown in Figure 19. It should be emphasized that this approach is highly debatable from the perspective of vertical load-bearing behavior, due to significant long-term deflections that have developed during the service life of the structures.

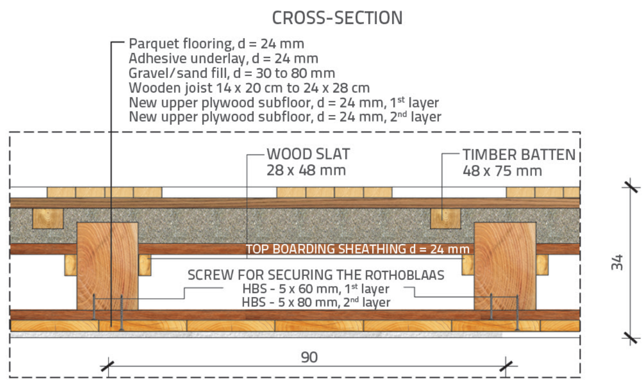

Figure 19. Strengthening of a floor structure with traditional timber beams and recessed infill by panel stiffening from the underside of the beams

Parisi and Piazza [28] conducted tests on floor structures with different types of strengthening, for which capacity curves were developed. In the unstrengthen structure, a midspan displacement of 50 mm occurs already at a resultant force of 80 kN, indicating that timber floor systems in their existing condition are highly flexible. By boarding the joists with two layers of planks, a midspan displacement of 45 mm is reached at a resultant force of 350 kN, while stiffness is further increased by adding perforated steel strips. The highest stiffness values in the conducted tests were achieved with three layers of veneer panels glued to the timber beams and with a timber–concrete composite slab, where a resultant force of 300 kN resulted in a midspan displacement of less than 5 mm.

3.2.2. Strengthening of beams using linear elements

Timber beams in floor structures are subjected to the self-weight of the structure and long-term permanent loads, which often result in long-term deflections. In order to rehabilitate and strengthen excessively deflected structures, as well as zones affected by excessive moisture or biological degradation, linear strengthening methods are applied in the form of additional timber elements or steel UPN profiles. These new elements are installed alongside the existing beams and connected using threaded rods or wood screws, thereby taking over part of the load from the original timber members. Installation is typically carried out on both sides of the beamst, and at the points where steel elements bear on the walls, it is necessary to use high-strength grout or mortar to ensure proper load transfer. Figure 20 illustrates a system in which UPN 200 steel profiles are installed on both sides. This type of strengthening is generally feasible only for floor structures without recessed infill, unless the floor layers are removed and the work is carried out from above rather than from the ceiling side.

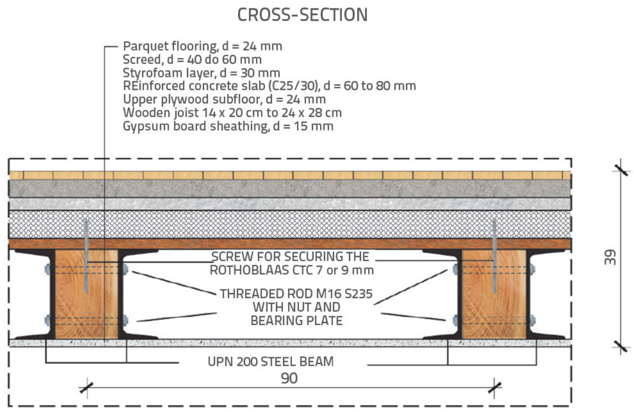

Figure 20. Strengthening of a timber beam using a steel UPN profile

4. Case studies

Following the analyses of construction materials, structural systems, and floor plan layouts, three types of buildings were examined through case studies as representative historic residential buildings of Zagreb. The types are described in Section 2.2, and a total of nine numerical models (three variants for each type) were developed using the 3Muri software [29]. In addition to the original structure analysed in its unstrengthen state, each characteristic floor plan configuration was also analysed with two strengthening variants: a timber laminated panel with a thickness of 6 cm and a composite reinforced concrete slab with a thickness of 8 cm. These numerical models were used to assess the influence of floor structures on the masonry system, without applying additional strengthening measures to vertical masonry elements, to ensure meaningful comparisons. The analysis was carried out only for the Significant Damage Limit State (SDLS), which is permitted when the structure is being upgraded to a maximum of Performance Level 2.

Mechanical properties of materials were not determined through in-situ testing but were adopted based on previous studies [16, 14, 30]. Based on the referenced literature and the available test results for buildings of similar age and structural systems, the mean masonry properties shown in Figure 21 were adopted for all numerical models. The modulus of elasticity used in the numerical analyses was 1500 N/mm², the shear modulus was 500 N/mm², and the assumed shear strength was 0.19 N/mm². It should be emphasized that these values represent assumptions made by the author to obtain comparable results across the numerical models. When designing structural strengthening measures in practice, however, it is essential to conduct in-situ testing of the mechanical properties of the masonry, and the values adopted in this study should not be applied uncritically. Furthermore, shear strength values reported in the literature are often below 0.15 N/mm².

The mechanical properties of timber used in the analyses correspond to solid softwood of strength class C20, considering the construction period and material degradation. For the strengthened variants, glued laminated timber of strength class GL24h and reinforced concrete of class C25/30 were used for the composite slab. Vertical loads acting on the structure were defined according to [31, 32]. The additional permanent load of the floor structure in the original condition is as follows: for the ceiling above the basement g_k = 5.34 kN/m²; for intermediate floors g_k = 3.48 to 4.29 kN/m² depending on thickness; and for the ceiling below the attic g_k = 3.08 to 3.51 kN/m² depending on whether the roof is pitched or flat. Seismic loading was defined according to [33, 34] with peak ground acceleration values of a_{475} = 0.25g for a 475-year return period and a_{95} = 0.13g for a 95-year return period.

Tabel 2. Mechanical properties of materials for all analyzed types

Mechanical properties	Brickwork
Elastic modulus, E [N/mm²]	1.500
Shear modulus, G [N/mm²]	500
w [kg/m³]	1.835
f_m [N/mm²]	4.00
f_k [N/mm²]	3.00
Shear strength, T [N/mm²]	0.19
f_{vlim} [N/mm²]	0.65
CF	1.35
γ_m	2.00
Shear drift	0.0053
Bending drift	0.0107
$\phi\ \infty$	0.0
Damage condition	Existing

Based on the defined geometry and loading for the three structural types, numerical models of the original state and strengthened variants were developed. The systems were discretized, and simplified macroelement models were generated, on which nonlinear analysis was performed using the N2 pushover method.

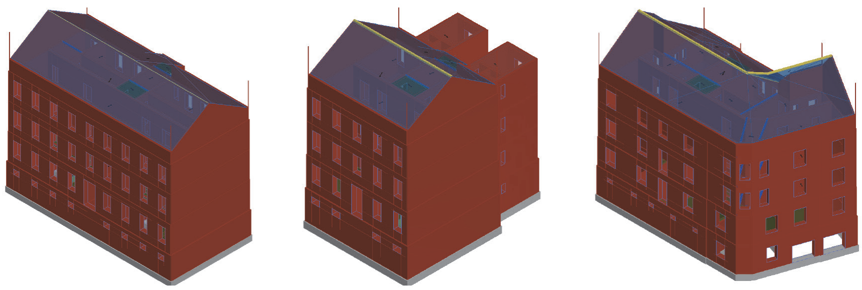

Figure 22. Numerical models for Type 1, Type 2, and Type 3 in the 3Muri software [29]

composite slab (Figure 25). An analysis of the floor plan layout and the orientation of floor structures in Type 2 shows that the floor systems are arranged in two directions, while the load-bearing walls in the courtyard zone are also oriented in two directions. Combined with the highly articulated floor plan geometry, this explains the observed distribution of activated masses.

4.1. Analysis and comparison of results

The analysis and comparison of results were carried out based on the total activated mass in the modes where the mass participation exceeded 5 %, considering the achieved structural displacement, as well as the calculated IZO coefficient, which,according to national regulations,represents the percentage of the structure's load-bearing capacity in comparison with contemporary design standards [35, 36].
The total activated mass of the Type 1 structure, for the original configuration as well as for the strengthened variants with timber and reinforced concrete composite slabs, is shown in Figure 23 for the x-direction and in Figure 24 for the y-direction.

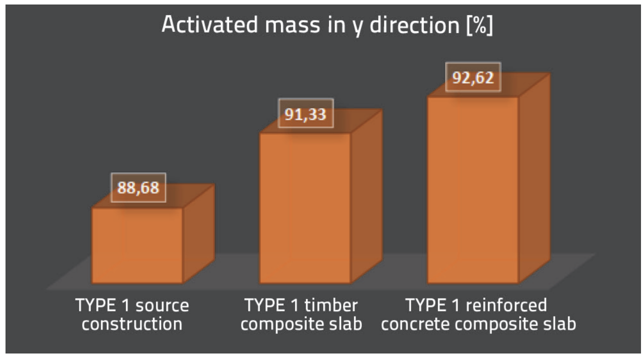

Figure 24. Sum of activated modal masses with participation greater than 5 % for the Type 1 structure in the Y direction

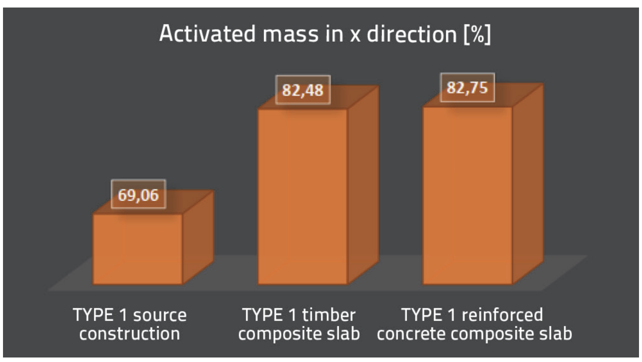

Figure 23. Sum of activated modal masses with participation greater than 5 % for the Type 1 structure in the X direction

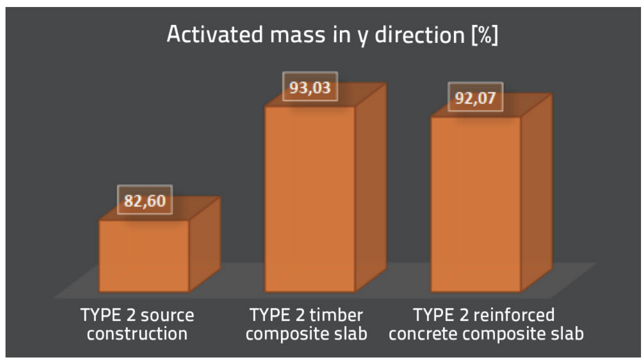

Figure 25. Sum of activated modal masses with participation greater than 5 % for the Type 2 structure in the Y direction

These figures indicate that the original structure exhibits the lowest activated mass in both directions. Improved structural connectivity is achieved with the timber composite slab, while the highest activated mass in both directions is recorded for the reinforced concrete composite slab. For the Type 3 structure, the total activated mass also increases with the application of the timber composite slab, while the highest level of structural connectivity is observed in the case of the reinforced concrete composite slab.
For the Type 2 floor plan configuration, the mass participation percentages in the x-direction follow trends similar to those observed for Type 1 and Type 3. However, in the y-direction, it is evident that the activated mass reaches 93.03 % for the structure strengthened with a timber composite slab and 92.07 % for the structure strengthened with a reinforced concrete composite slab (Figure 25). An analysis of

Based on all available data, it is evident that the two considered strengthening methods provide improved structural connectivity, increase the participation of a larger percentage of mass, and thus directly contribute to preventing the activation of local masonry failure mechanisms, i.e., out-of-plane wall failure. These interventions represent a necessary step in seismic strengthening and cannot be replaced solely by strengthening vertical elements.
The calculated IZO coefficients indicate that the load-bearing capacity of the structure significantly increases through improved connectivity at floor levels, particularly for Type 1 in the x-direction, since this layout lacks walls that extend across the full length of the building in that direction (Figures 26 and 27).

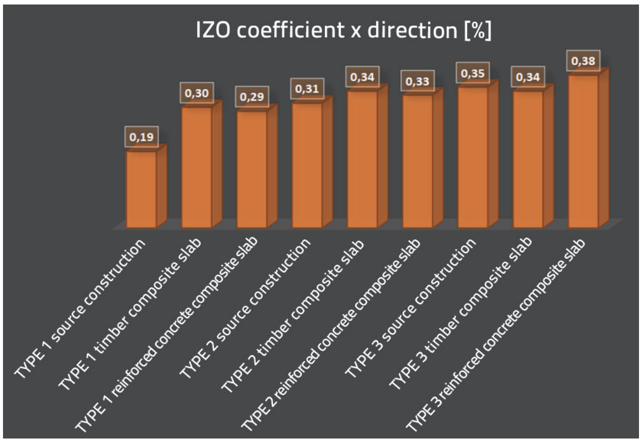

Figure 26. IZO coefficients of structures for the X direction

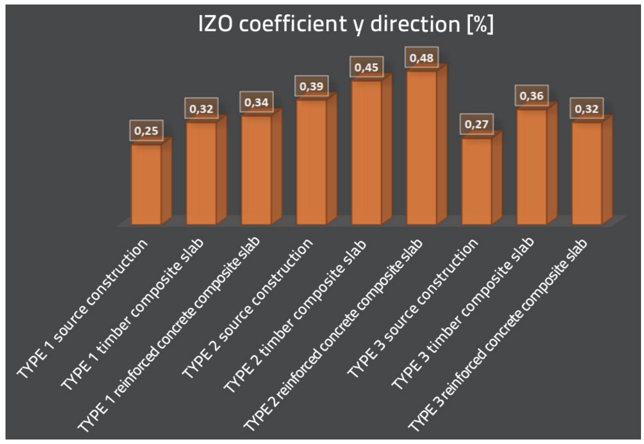

Figure 28. IZO coefficients of structures for the Y direction

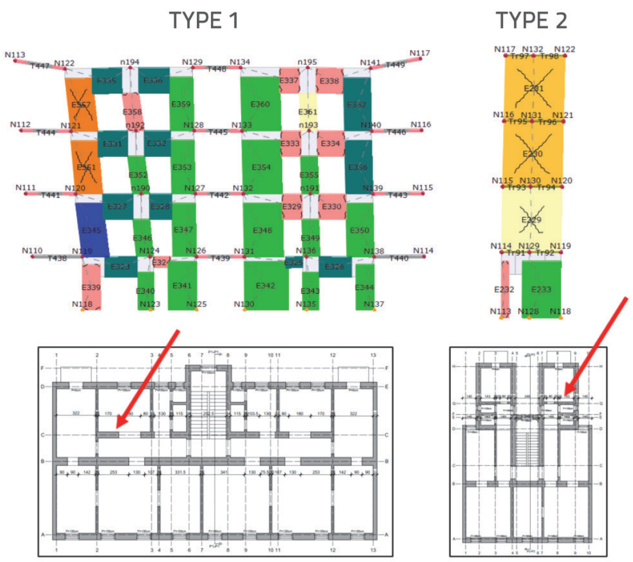

Figure 27. Critical positions for the original state of Type 1 and the strengthened state with a timber slab for Type 2 in the X direction

It can be observed that the load-bearing capacity of the structure increases by approximately 7 % to 10 % for both the timber composite slab and the reinforced concrete composite slab, even without strengthening the vertical elements. For Type 1 and Type 2, the timber composite slab provides lower capacity compared to the reinforced concrete composite slab. However, for Type 3, the timber composite slab achieves 36 % of the load-bearing capacity, while the reinforced concrete composite slab reaches 32 %, in comparison with contemporary Eurocode requirements. For all types and strengthening methods, the same wall locations reach significant damage states; however, the strengthened and better-connected structure allows for larger displacements and a more favorable distribution of seismic forces. The initial unconnected structure, particularly in the y-direction, typically consists only of thin, unconnected partition walls, which do not support floor structures. By improving connectivity in the y-direction through interventions in the floor structures, an increase in load-bearing capacity relative to modern standards is generally achieved. However, the full structural capacity can only be realized through the strengthening of vertical masonry walls in combination with floor structure interventions. As a result, very short walls enter a state of significant damage at relatively small displacements. For other floor plan configurations, even with strengthening measures, there is no substantial increase in load-bearing capacity. This is because the capacities of vertical elements are already exceeded, and despite improved connectivity and a more favorable distribution of inertial forces, very high stresses still occur in critical zones. It should also be noted that timber joists are predominantly supported by longitudinal walls (x-direction), which results in significant axial compressive forces in the walls, having a favorable effect when checking shear resistance. Figure 28 presents the values of the IZO coefficient for all analysed structural types and strengthening methods.

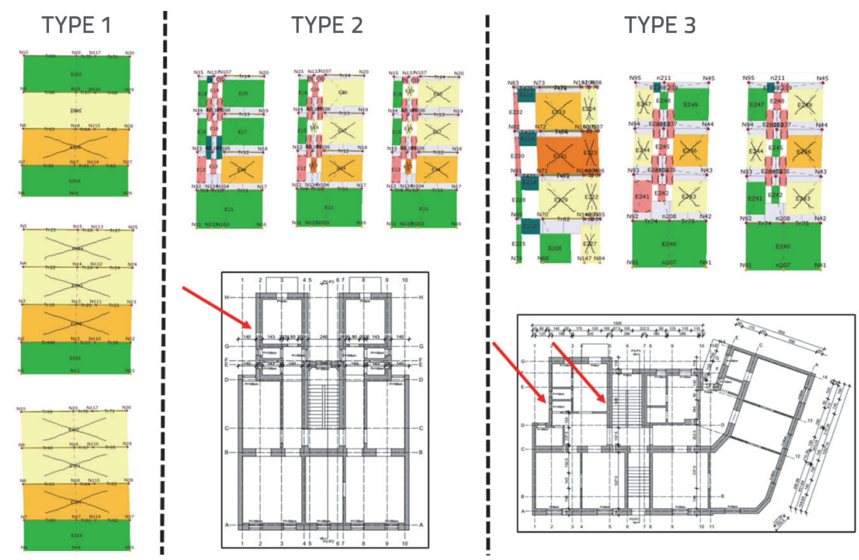

Figure 29. Critical positions for the three structural types in the original and strengthened states

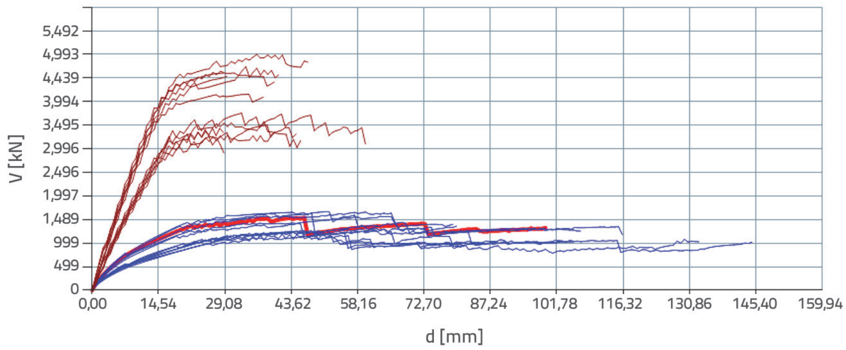

Figure 30. Capacity curve for 24 analyses – Type 2 (original state)

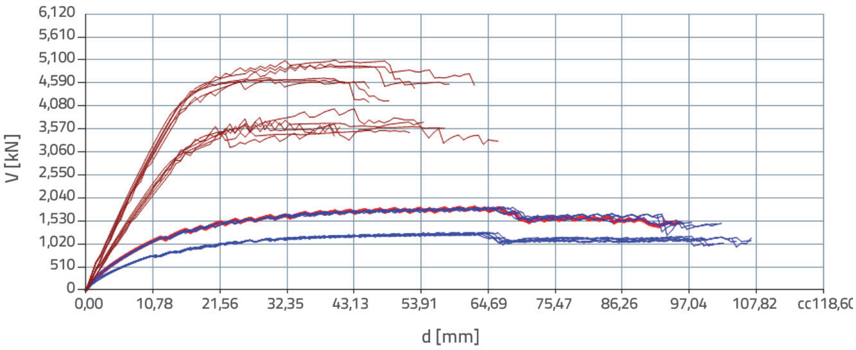

Figure 31. Capacity curve for 24 analyses – Type 2 (strengthened with an RC composite slab, d = 8 cm)

Analysis of the capacity curves shown in Figures 30 and 31 for Type 2 indicates that strengthening with a reinforced concrete composite slab leads to an increase in both displacement capacity and shear force in the y-direction. In contrast, for the x-direction, the increase in shear force and overall structural capacity is negligible.

5. Conclusion

This paper systematically analyses timber floor structures as the dominant type of floor systems in unreinforced masonry buildings of traditional (Austro-Hungarian) construction in Zagreb. Their typology, structural characteristics, and influence on the global seismic behavior of buildings were examined. It has been shown that such systems, due to their limited in-plane stiffness and insufficient connection with load-bearing walls, do not ensure effective distribution of horizontal actions, resulting in an unfavorable spatial response of the structure. Consequently, timber floor systems represent one of the main sources of seismic vulnerability in the observed building stock. The analysis of typical floor plan layouts and the relationship between floor and vertical load-bearing elements further confirms that the absence of "box" behavior, characteristic of modern masonry systems with reinforced concrete slabs, has a significant impact on the level of damage. In this context, the most common strengthening methods for floor structures were considered, primarily in the form of timber and reinforced concrete composite slabs. These interventions increase in-plane stiffness, improve the connectivity of structural elements, and enable a more uniform distribution of seismic forces, while activating a larger portion of the system mass. However, strengthening only the timber floor structures cannot bring a building to the required level of rehabilitation, i.e., the target IZO coefficient according to [35-37] and strengthening of vertical load-bearing elements is necessary. Likewise, interventions limited only to vertical elements will not achieve the desired performance, as higher-intensity earthquakes would still trigger local out-of-plane failure mechanisms of masonry. This clearly confirms the need for an integrated strengthening approach that includes both floor systems and vertical load-bearing elements, ensuring their effective interaction. In the post-earthquake reconstruction of Zagreb, a pronounced trend has been observed of removing and replacing timber structural elements, particularly in roof structures, where traditional systems are systematically replaced with new materials [38]. Such practice reflects a clear inconsistency in the approach to rehabilitation: while masonry components are regularly treated as valuable heritage to be preserved, timber, despite its clearly defined role as a structural material, is often neglected and unnecessarily removed. This overlooks the fact that timber structures, with appropriate rehabilitation and targeted strengthening, can meet modern requirements of load-bearing capacity and serviceability. As a result, timber elements frequently end up as waste (Figure 32), leading not only to the loss of material value but also to the loss of the original structural logic and authenticity of the system.

Figure 32. Dismantled timber primary load-bearing elements during structural rehabilitation

LITERATURA

[1] Kolar, M.: Urbanistički razvoj Zagreba tijekom 17. i 18. stoljeća, Zagreb, 2014.

[2] Kolar, M.: Urbanistički razvoj Zagreba tijekom 17. i 18. stoljeća, dostupno na: https://darkoantolkovic.wordpress.com/2019/01/24/urbanisticki-razvoj-zagreba-tijekom-17-i-18-stoljeca, pristupljeno: 16. travnja 2026.

[3] Uroš, M., Todorić, M., Crnogorac, M., Atalić, J., Šavor Novak, M., Lakušić, S.: Potresno inženjerstvo – Obnova zidanih zgrada, Građevinski fakultet Sveučilišta u Zagrebu, Zagreb, 2021.

[4] Šavor Novak, M., Uroš, M., Atalić, J., Herak, M., Demšić, M., Baniček, M., Lazarević, D., Bijelić, N., Crnogorac, M., Todorić, M.: Potres u Zagrebu 22. ožujka 2020. – preliminarni izvještaj o seizmološkim istraživanjima i oštećenjima zgrada, GRAĐEVINAR, 72 (2020) 10, pp. 843–867.

[5] Moretić, A., Chieffo, N., Stepinac, M., Lourenço, P.B.: Vulnerability assessment of historical building aggregates in Zagreb: implementation of a macroseismic approach, Bulletin of Earthquake Engineering, 21 (2023).

[6] Stepinac, M., Ožić, K., Ninčević, A., Funari, M.F., Lourenço, P.B.: A digital tool based on code indicators to assess the seismic behaviour of existing masonry structures: Application to Croatian residential buildings, Structures, 58 (2023), pp. 1–10.

[7] HRN EN 1990: Eurokod – Osnove projektiranja konstrukcija, Hrvatski zavod za norme, Zagreb, 2011.

[8] HRN EN 1990:2011/NA: Eurokod – Osnove projektiranja konstrukcija – Nacionalni dodatak, Hrvatski zavod za norme, Zagreb, 2011.

[9] Jajčević, K., Demšić, M., Šavor Novak, M., Atalić, J., Uroš, M.: Prevrtanje nepridržanih zidova – prevladavajući mehanizam oštećenja starih zidanih zgrada, GRAĐEVINAR, 77 (2025), pp. 1057–1069.

[10] Kahraman, O., Kaçın, S.: Detaljna analiza šteta nastalih u pokrajini Hatay uslijed potresa u Pazarcıku i Elbistanu u veljači 2023., GRAĐEVINAR, 77 (2025) 10, pp. 877–887.

[11] Čaušević, A., Rustempašić, N.: Rekonstrukcije zidanih objekata visokogradnje, Univerzitet u Sarajevu, Sarajevo, 2014.

[12] Boynton, R.S.: Chemistry and Technology of Lime and Limestone, John Wiley & Sons, New York, 1996.

[13] Wingate, M.: Small-scale Lime Burning: A Practical Introduction, Intermediate Technology Publications, London, 1985.

[14] Ortega, J.H., Stepinac, M., Lulić, L., Nunez, G.M., Saloustros, S., Aranha, C., Greco, F.: Correlation between sonic pulse velocity and flat-jack tests for the estimation of the elastic properties of unreinforced brick masonry: Case studies from Croatia, Case Studies in Construction Materials, 18 (2023).

[15] Lulić, L.: Određivanje mehaničkih svojstava nearmiranog ziđa od pune opeke nerazornim i polurazornim metodama, doktorski rad, Građevinski fakultet Sveučilišta u Zagrebu, Zagreb, 2024.

[16] Stepinac, M., Lulić, L., Damjanović, D., Duvnjak, I., Bartolac, M., Lourenço, P.B.: Experimental Evaluation of Unreinforced Brick Masonry Mechanical Properties by the Flat-Jack Method – an Extensive Campaign in Croatia, International Journal of Architectural Heritage, 18 (2024) 4, pp. 691–708.

[17] Cruz, H., Yeomans, D., Tsakanika, E., Macchioni, N., Jorissen, A., Touza, M., Mannucci, M., Lourenço, P. B.: Guidelines for On-Site Assessment of Historic Timber Structures, International Journal of Architectural Heritage, 9 (2015) 3, pp. 277–289.

[18] Dietsch, P., Kreuzinger, H.: Guideline on the assessment of timber structures, Engineering Structures, 33 (2011) 11, pp. 2983–2986.

[19] Stepinac, M., Rajčić, V., Barbalić, J.: Pregled i ocjena stanja postojećih drvenih konstrukcija, GRAĐEVINAR, 69 (2017) 9, pp. 861–873.

[20] Perković, N., Stepinac, M., Rajčić, V., Barbalić, J.: Assessment of Timber Roof Structures before and after Earthquakes, Buildings, 11 (2021) 23.

[21] Čizmar, D., Volarić, I.: Projekt sanacije krovne konstrukcije skladišta kalijevog klorida, projektna dokumentacija, Zagreb, 2019.

[22] 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, International Journal of Disaster Risk Reduction, 56 (2021), Article 102140.

[23] Krause, M., Kurz, J.H., Lanata, F., Krstevska, L., Cavalli, A.: Needs for further developing monitoring and NDT methods for timber structures, in: Proceedings of the International Conference on Structural Health Assessment of Timber Structures, Wrocław, Poland, 2015.

[24] Stepinac, M., Rajčić, V., Barbalić, J.: Utjecaj dugotrajnog opterećenja na spregnute sustave drvo–beton, GRAĐEVINAR, 67 (2015) 3, pp. 235–246.

[25] Daanoune, N., Kernou, N., Fellah, M., El-Hiti, G.A.: Pouzdanost i mehanička svojstva spregnutih nosača drvo–beton u nelinearnome području ponašanja, GRAĐEVINAR, 76 (2024) 10, pp. 993–1003.

[26] Čizmar, D., Vrančić, M.: Proračun spregnutih nosača drvo–beton, GRAĐEVINAR, 68 (2016) 7, pp. 559–570.

[27] Mirra, M.: Seismic Behaviour of Masonry Buildings with Timber Diaphragms, Delft University of Technology, Delft, 2022.

[28] Parisi, M.A., Piazza, M.: Seismic strengthening and seismic improvement of timber structures, Construction and Building Materials, 2015.

[29] STADATA: 3MURI Project, dostupno na: https://stadata.com/en/3-muri-project/, pristupljeno: 16. travnja 2026.

[30] Krolo, J., Damjanović, D., Duvnjak, I., Frančić Smrkić, M., Bartolac, M., Košćak, J.: Metode određivanja mehaničkih svojstava ziđa, GRAĐEVINAR, 73 (2021) 2, pp. 127–140.

[31] HRN EN 1991-1-1: Eurokod 1 – Djelovanja na konstrukcije – Dio 1-1: Opća djelovanja – Gustoće, vlastite težine, korisna opterećenja zgrada, Hrvatski zavod za norme, Zagreb, 2012.

[32] HRN EN 1991-1-1:2012/NA: Eurokod 1 – Djelovanja na konstrukcije – Dio 1-1: Nacionalni dodatak, Hrvatski zavod za norme, Zagreb, 2012.

[33] HRN EN 1998-1: Eurokod 8 – Projektiranje potresne otpornosti konstrukcija – Dio 1: Opća pravila, potresna djelovanja i pravila za zgrade, Hrvatski zavod za norme, Zagreb, 2011.

[34] HRN EN 1998-1:2011/NA: Eurokod 8 – Projektiranje potresne otpornosti konstrukcija – Dio 1: Opća pravila, potresna djelovanja i pravila za zgrade – Nacionalni dodatak, Hrvatski zavod za norme, Zagreb, 2011.

[35] Tehnički propis o izmjenama i dopunama Tehničkog propisa za građevinske konstrukcije, Narodne novine, Zagreb, 2020.

[36] Tehnički propis o izmjenama i dopunama Tehničkog propisa za građevinske konstrukcije, Narodne novine, Zagreb, 2022.

[37] Tehnički propis za građevinske konstrukcije, Narodne novine, Zagreb, 2017.

[38] Volarić, I., Čizmar, D., Serdarević, Š., Bakmaz, I.: Klasična drvena krovišta povijesnih građevina u Zagrebu, Polytechnic and Design, 13 (2025) 2, pp. 98–112.