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ASSESSMENT OF A HISTORIC MASONRY BUILDING: A CASE STUDY OF THE 1ST GYMNASIUM

Keywords

Austro-Hungarian buildings
drone inspection
infrared thermography
seismic assessment
unreinforced masonry structures

Abstract

In this research, a rapid seismic assessment of the I Gymnasium in Sarajevo (Bosnia and Herzegovina) was conducted. Structural information was obtained from original designs, visual inspection, and supported by the usage of a drone and infrared thermography. The vulnerability of the structure was assessed by implementing the Vulnerability Index Method (VIM), producing a vulnerability curve that quantifies mean damage grades for various seismic intensities according to the EMS-98 scale, enabling the quantification of risk for various seismic scenarios. The structure would experience no to minor damage if exposed to weaker earthquakes (IV–V), while intensities VII–VIII, relevant for Sarajevo, lead to moderate damage with mean grades of 1.86 and 2.93 and increasing probabilities of very severe damage and collapse (≈24% and 2% at VIII). For earthquakes of higher intensities (X–XII), there is a significant increase in heavy and severe damage, leading to the complete collapse in the case of XII intensity level (mean grade 4.89). While this assessment focuses on a single building, its reinforced concrete frame with masonry infill represents a common typology in Sarajevo, making the results broadly relevant for similar educational structures. The paper further discussed the effects that have a direct impact on the degradation of masonry material, leading to a reduction of its resilience. An aspect of inadequate strengthening is emphasized, together with the importance of regular maintenance for prolonging the life of these types of structures.

INTRODUCTION

The period of Austro-Hungarian rule in Bosnia and Herzegovina (1878-1918) marked a significant transformation in the region's architectural and construction practices, with the introduction of contemporary principles of urban planning, infrastructure development, and European architectural styles. During this period, structures were made of masonry, either stone or brick load-bearing walls, of various thicknesses, while the floors were made of timber, representing flexible floors. At that time, no seismic rules existed, so most of these structures were constructed without any seismic regulations.

This region became a part of the Austro-Hungarian Empire, so construction practices changed significantly in relation to the Ottoman Empire; however, certain local features were kept. This approach led to the construction of numerous public buildings, residential houses, and administrative buildings that reflected a combination of European historical styles and traditional Ottoman influences. Masonry constructions with wooden ceilings became a recognizable sign of this period due to their adaptability and the availability of materials.

Bosnia and Herzegovina is located within a seismically active region influenced by the complex interaction between the Adriatic microplate and the Eurasian plate. Moderate to strong earthquakes have been recorded throughout history, highlighting the built environment's seismic vulnerability. In particular, the 1969 Banja Luka earthquake caused significant damage to masonry buildings, exposing their inadequate seismic resistance and leading to the first systematic implementation of seismic design regulations in the former Yugoslavia. Similarly, the 1979 Montenegro earthquake had widespread regional impacts, further emphasizing the need for improved seismic assessment methodologies.

A large portion of the existing building stock, including structures constructed during the Austro-Hungarian period, was designed without consideration of seismic actions. These buildings are typically characterized by unreinforced masonry walls, flexible diaphragms, and insufficient connections between structural elements, making them particularly vulnerable to seismic loading. Therefore, their assessment represents a critical task within the broader framework of seismic risk mitigation in Southeast Europe.

In this paper, the seismic vulnerability of a single school building from the Austro-Hungarian period has been assessed. The analysis focuses exclusively on this building, examining its historical construction characteristics, structural typology, and potential damage level due to various macro seismic intensities of earthquake.

STRUCTURAL ELEMENTS OF MASONRY STRUCTURES

Elements made of stone and fired bricks were most frequently used for the construction of the load-bearing walls in masonry structures, and lime mortar as a binder. This type of construction has several benefits, including fast construction, good thermal insulation, important for the climate conditions of this region. Due to higher compressive strength, stone was often used for the construction of load-bearing walls at the lower floors, and the upper floors were usually made of fired brick. The dimensions of the bricks during the Austro-Hungarian era were approximately 30cmx15cmx7cm (Ademović, 2024), which was larger than the standard bricks (25 cm x 12 cm x 6.5 cm) used today.

The thickness of the walls often varied, with public buildings having more massive walls for increased durability and protection. The outer walls of public and administrative buildings could have reached a thickness of 50 cm and up to 1 meter, especially on the lower floors, due to the need for structural stability and thermal insulation. On higher floors, the thickness of the walls was smaller, around 30-40 cm. The inner walls of residential buildings and less massive buildings were around 25-30 cm thick, while partition walls had dimensions of 7 to 15 cm. The walls surrounding the staircase and the elevator in the basement are 60 and 45 cm thick, whereas all other floors have their thickness reduced by 15 cm. The wall thickness was prescribed in the Viennese building code (Kolbitsch, 1989: 15).

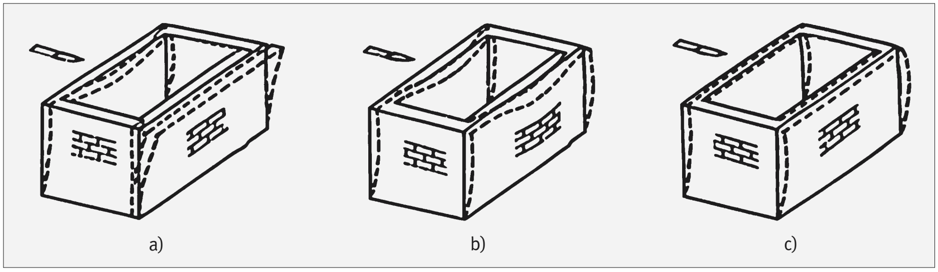


Fig. 1 Influence of connection and stiffness of floor on the “box” model: a) flexible floor without horizontal bond beams; b) flexible floor with horizontal bond beams; c) rigid floor with horizontal bond beams (Gallonelli, 2007: 57)

SEISMIC BEHAVIOUR OF STRUCTURES WITH RESPECT TO FLOOR TYPES

Timber was extensively used for floor constructions due to its availability and ease of construction. Floors were typically supported by wooden beams embedded in the stone walls, and wooden planks formed the walking surface (Fig. 2a). In some cases, the beams were reinforced with iron elements to increase their resistance and longevity. Although there were no universal standards as they exist today, certain characteristics were typical of wooden ceilings of that time. The typical dimensions of these beams were 12 to 20 cm x 20 to 30 cm (Peulić, 2022: 512), depending on the length and load-bearing capacity required for the construction. Beams were typically placed at a distance of 60-100 cm, depending on the load and the room’s size. The ceiling height was on average between 2.5 and 4 m, but in administrative or larger public buildings, the ceiling height could reach larger dimensions, even up to 5 m in some cases. For older buildings with larger spans Peulić, 2022: 512), such as schools and halls, it was recommended to retain double-beamed ceilings. Although larger beams had a larger volume, they did not completely eliminate vibrations.

A common practice in seismic analysis of buildings is to assume that diaphragms, or floors, are rigid. This assumption is considered acceptable for reinforced concrete floors, which, due to their massive characteristics and strength, can effectively transfer seismic forces to vertical elements. This type of action is known as “box action”. However, this assumption does not apply to wooden floors, which, due to their relatively lower mass and elastic properties, behave differently from reinforced concrete floors. For buildings with wooden floors, it is necessary to carry out specific analyses that take these characteristics into account.

A floor structure is considered flexible when the maximum lateral deformation of the diaphragm is twice the average displacement of the floor. This can be confirmed by comparing the calculated displacement of the midpoint in the floor plane under lateral load with the displacement in adjacent vertical elements under the same lateral load (Elsherbeny, 1999: 11). Unlike rigid floors, flexible floors cannot redistribute forces resulting from torsional deformations of the structure. This inability to transfer the forces can lead to serious problems and structural failure, because the forces are not distributed evenly throughout the structural elements. On the other hand, rigid floors better distribute torsional effects to vertical elements, with the distribution being proportional to the stiffness of these elements.

Thus, in older masonry buildings with flexible floors without horizontal bracing, and in which the wooden beams are not firmly connected to the walls, the walls separate from each other during an earthquake. Due to the lack of adequate bonding, vertical cracks form at the joints of the walls, especially on the end walls, i.e., those perpendicular to the direction of the earthquake. These cracks are caused by the walls bending out of plane, which is a type I failure mechanism.

Additionally, cracks appear at the ends of walls parallel to the direction of earthquake action due to their limited tensile strength, which is not sufficient to transfer inertial forces. In such conditions, uncoordinated movement of individual walls occurs, which increases the overall vulnerability of the structure. The outer walls are particularly susceptible to overturning and breaking, which further jeopardizes the stability of the building.

Figure 1 shows the influence of the connection between the floors and walls and the stiffness of the floors on the seismic action of the structure. The variants shown illustrate different seismic response scenarios depending on the degree of interconnection of structural elements.

Fig. 1a illustrates a structure with a flexible floor without a rigid connection between orthogonal walls. In such a case, each wall acts as a separate element, without the possibility of working together with other load-bearing walls. This configuration significantly weakens the resistance to horizontal forces. As the walls are not mutually supported, this increases the risk of a local or global collapse during seismic loading. Fig. 1b shows a construction in which the walls are connected by

Fig. 2 Inadequate strengthening methods (Binda and Modena, 2011)

horizontal bond beams at the junction with the floor, while the floor remains flexible. This solution somewhat improves seismic resistance because horizontal bond beams connect the walls and enable their partial associated deformation. It is what reduces the possibility of complete separation of the walls during an earthquake. In this type of construction, the significant out-of-plane bending of the walls is still present, which may lead to local collapse mechanisms. The construction of the rigid floors and horizontal bond beam is illustrated in Fig. 1c. This changes the action of the structure once exposed to an earthquake. Now the structure overall demonstrates “box action,” which is more favourable. In this case, horizontal forces are efficiently transferred onto the walls with respect to their stiffness.

The recognized dominant problem in all combinations of flexible floors is the possibility of out-of-plane failure due to the fact that in these structures, walls are not mutually connected, and they are also not connected to the floors, leading to significant relative movements between individual elements.

The case study building (1st Gymnasium) is characterized by a masonry structural system with timber floor diaphragms. The load-bearing system consists of longitudinal and transverse masonry walls with variable thickness, forming a predominantly regular plan layout. Openings constitute approximately 17.55% of the façade surface, locally reducing wall stiffness and load-bearing capacity.

The floor system is composed of timber beams with plank decking, acting as flexible diaphragms with limited in-plane stiffness. No effective horizontal bracing or diaphragm stiffening elements were identified, and connections between floors and walls are assessed as insufficient to ensure composite action. Consequently, diaphragm continuity is limited, and load transfer between orthogonal walls is not fully achieved.

Inadequate strengthening of unreinforced masonry structures (URM) may lead to catastrophic scenarios. An example of this was seen, after the earthquake in the Friuli region, where improper interventions on historical buildings caused the so-called *hammering effect*. In particular, the installation of reinforced concrete slabs significantly altered the dynamic response of the structure. The increased stiffness of the floor led to the concentration of seismic forces in the walls, which resulted in out-of-plane failure (see Fig. 2).

There is empirical evidence from recent regional seismic events, specifically the 2020 Zagreb and Petrinja earthquakes, which prominently affected the URM buildings. The 2020 Zagreb earthquake struck the city’s urban area, resulting in extensive damage to historic URM buildings, including school facilities and cultural heritage buildings, despite moderate seismic intensity levels. Many buildings experienced cracking, partial façade collapses, and other forms of structural degradation, highlighting the seismic vulnerability of traditional masonry construction without adequate diaphragms, ties, or retrofitting measures (Arbutina et al., 2025; Stepinac et al., 2023; Lulić et al., 2021).

Later that same year, on 29 December 2020, a Mw 6.4 earthquake occurred near Petrinja, causing widespread damage in Sisak-Moslavina County and surrounding areas, with URM buildings being among the most severely affected structural types. Post-earthquake reconnaissance and damage assessments reported significant structural failures in schools and other public buildings, underlining the limitations of conventional strengthening techniques such as rigid insertions or poorly detailed infill repairs.

EDUCATIONAL INSTITUTIONS

In Bosnia and Herzegovina, several schools are situated in buildings constructed during the Austro-Hungarian period, which still serve their original purpose today. Some of the examples are:

First Gymnasium (High School) in Sarajevo: Founded in 1879, this is the oldest secular high school in Bosnia and Herzegovina. It was originally located in other locations; however, in 1891, it was moved to a new building built in neo-Renaissance style (Fig. 3a).

Elementary School “Safvet-beg Bašagić”: The building of this school was constructed between 1890 and 1892, specifically for educational purposes (Fig.3b).

Sarajevo High School of Applied Arts: This school, founded during the Austro-Hungarian administration, is in a building built in 1893 (Fig. 3c).

Fig. 3 a) I Gymnasium; b) Safvet-beg Bašagić Elementary School; c) High School of Applied Arts; High School of Agriculture, Food, Veterinary Medicine and Services

The building of the public institution *“High School of Agriculture, Food, Veterinary Medicine and Services”* (Fig.3d) is located across the street.

All these structures date from the Austro-Hungarian period and display the characteristic architectural features of that era, marked by rapid urbanization and the construction of many public and educational buildings in modern architectural styles.

For this study, the focus is limited to a single building, the First Gymnasium in Sarajevo (Fig. 3a), selected due to its historical significance, representative construction characteristics, and the fact that only this building was a part of the project: “*Austro-Hungarian Educational Buildings and their Sustainability in Specific Conditions, with Special Reference to the First Gymnasium of Sarajevo*“ The other institutions are mentioned here solely to provide contextual background on the typology and architectural trends of Austro-Hungarian school buildings in the region.

METHODOLOGICAL FRAMEWORK AND VULNERABILITY INDEX

In studies assessing the seismic vulnerability of school buildings, the vulnerability index is often used as a quantitative measure of their sensitivity to seismic impacts. Nonlinear parametric analysis, which allows the calculation of the Seismic Vulnerability Index (SVI), or the Vulnerability Index Method (VIM), which is based on the statistical relationship between macro seismic intensity (e.g., according to the EMS 98 scale) and actual observed damage to buildings (Benedetti and Petrini, 1984), GNDT (1993). In this model, parameters are defined (e.g., type of construction, irregularities in the floor plan, number of floors, age of the building, etc.) to which weight values are assigned. By combining these parameters, the final vulnerability index is calculated.

In Europe, the RISK-UE (Milutinovic and Trendafiloski, 2003) significantly contributed to the development of simplified methodologies for assessing seismic vulnerability. These approaches use vulnerability indices based on key building parameters, such as typology, materials, number of floors, and geometric irregularities, to classify buildings into predefined vulnerability classes. The methodologies are suitable for large-scale assessments when detailed engineering analyses are not feasible. For example, in Mostaganem (Algeria), the RISK UE methodology was applied to 516 school buildings, generating vulnerability curves and damage scenarios according to EMS 98 intensities (Bendehiba et al., 2023: 7).

In the specific case, seismic assessment of the I Gymnasium was carried out in accordance with the RISK-UE Methodology (Milutinovic and Trendafiloski, 2003), taking into account several parameters, such as structural typology, including the type of the floor, location of the building, number or floors, roof type, layout dimensions, soil morphology, complex structure, maintenance, irregularity in height and plan, and a vulnerability curve was obtained (Fig. 4).

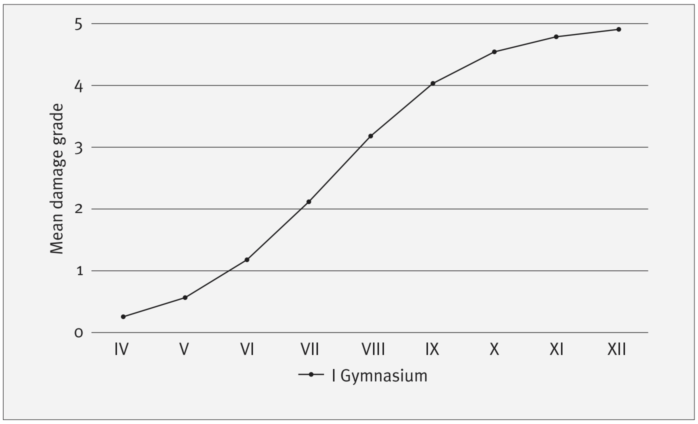
The vulnerability index, V_I, for a building is obtained according to the well-known equation (1), where V_I°, the vulnerability index, depends on the building’s typology and the seismic action modifier ΔV_M, into account, which takes into account the above mentioned parameters.

$$V_I = V_I^{\circ} + \Delta V_M \quad (1)$$

The mean damage grade μ_D, representing the average level of physical damage (in this case, only for this structure) when exposed to a specific earthquake intensity, is obtained using equation (2), primary employed in the macro seismic method

$$\mu_D = 2.5\left[1+\tanh\left(\frac{I+6.25V_I-13.1}{Q}\right)\right] \quad (2)$$

Where I represent the intensity according to the EMS-98 scale, and Q represent the ductility factor taken as 2.3 for this masonry structure.



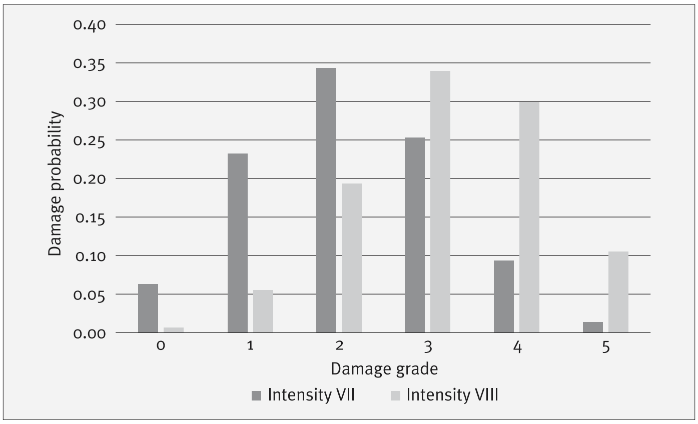


Fig. 4 Vulnerability curve for the building of the I Gymnasium

Fig. 5 Damage probability matrix

The vulnerability curve illustrates the relationship between the expected mean damage grade of the building and the earthquake's intensity as per the EMS 98 scale. Structure action is assessed for different earthquake intensities, from the weakest (IV) to the most catastrophic earthquake (XII). According to the assessment, the building where the I Gymnasium is located would experience small damage if exposed only to earthquakes of lower intensity (IV), keeping the mean damage grade lower than 0.5, indicating superficial or negligible damage to the structure and facades. The slope of the vulnerability curve gradually increases as the earthquake intensity rises. The most interesting intensities for Sarajevo are VII and VIII, whereby the mean damage grade reaches values of 2.12 and 3.19, indicating moderate damage, which would be manifested with cracks in the non-load-bearing walls and potential damage to load-bearing elements. At intensity VII, the building shows moderate resilience, with the highest probabilities in moderate damage (≈34.3%) and substantial to heavy damage (≈25.3%). As the intensity rises to VIII, the probability of substantial to heavy damage increases sharply to ≈34%, and very serious damage would be experienced by 30% of all structures, reflecting a significant escalation of vulnerability with stronger ground shaking. This is following the observed damages in the Croatian schools of this typology (Salaman et al., 2023: 10, 15; Lulić et al. 2021: 2; Atalić et al. 2023). These results highlight the building's sensitivity to higher-intensity earthquakes and emphasize the importance of targeted risk reduction measures, structural strengthening, and informed seismic risk management strategies (Fig. 5). The quantified probabilities provide a reliable basis for planning targeted risk reduction measures, structural strengthening, and informed seismic risk management. The main limitation of this analysis is that it does not account for the strengthened floor; a more detailed assessment incorporating this element will be conducted in subsequent analyses.

Fig. 6 Moisture penetration, cracking, and peeling of the material (author's photos)

For earthquakes of higher intensities (X–XII), the curve shows a rapid, sharp increase, indicating a significant increase in heavy and severe damage, leading to the complete collapse in the case of XII intensity level, where the mean damage grade reaches the value of 4.91. This kind of information is very valuable as it identifies earthquake intensities when the building becomes particularly vulnerable. Useful information for engineers and planners when in the processes of decision-making with respect to strengthening, rehabilitation, and seismic safety of educational facilities.

Degradation of masonry structures

Factors that have a direct influence on the degradation of masonry structures are moisture and water penetration, chemical reactions, biological activity, and mechanical impacts. These elements can accelerate the degradation of the material and reduce the load-bearing capacity of these structures. For that reason, it is of the utmost importance to understand this process and use this knowledge for adequate maintenance, and if needed, the strengthening of masonry structures.

Moisture penetration

One of the most common causes of masonry deterioration is the penetration of moisture into walls (see Fig. 6), which can have various negative consequences. Water penetrating the walls carries soluble salts that crystallize on the surface, weakening the mortar and reducing the building's aesthetic value.

In colder areas, freeze-thaw cycles of water absorbed into the walls can cause cracking and peeling of the material (see Fig. 6), especially in porous walls such as limestone or sandstone. Incomplete or inadequate drainage can lead to moisture rising through the walls due to capillary action, causing the rotting of wooden structural members, mold growth, and deterioration of stone or brick walls, reducing the durability and safety of the building.

The aerial and thermographic documentation was carried out using a DJI Mavic 3T drone equipped with an integrated thermal imaging system. The thermal sensor is based on an uncooled VOx microbolometer with a resolution of 640×512 px, 30 Hz frame rate, 12 μm pixel pitch, and NETD ≤ 50 mK, which makes it suitable for the non-contact detection of surface temperature differentials potentially associated with moisture ingress, detachments, fissures, and other material discontinuities. The platform also ensured stable flight performance and reliable positioning, enabling inspection of elevated and otherwise difficult-to-access areas. The survey was performed through three separate recording sessions during winter temperature inversion conditions, with the interior space additionally heated to enhance the thermal gradient between interior and exterior surfaces. Recordings were undertaken during three different parts of the day, each with a duration of approximately 10 minutes. This repeated acquisition strategy was adopted to improve the consistency of thermal observations and to reduce the possibility of interpreting isolated or transient anomalies as representative defects. During image acquisition, the thermal camera operated in automatic calibration mode, which was maintained as the standard survey setting.

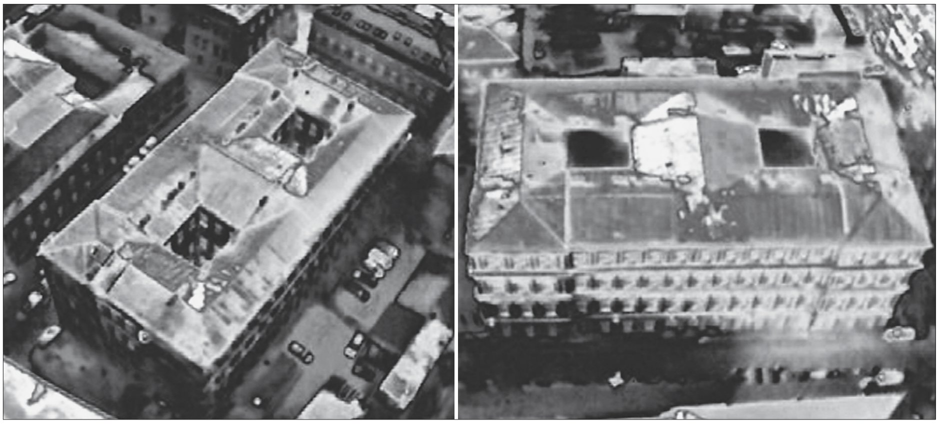


Fig. 7 Infrared thermography method of the I Gymnasium (author's photos)

The conditions of the building envelope were clearly revealed by infrared thermography. Zones with higher temperatures are marked in colours going from yellow, orange to green, while areas with lower temperatures are marked with cyan and dark blue. Locations

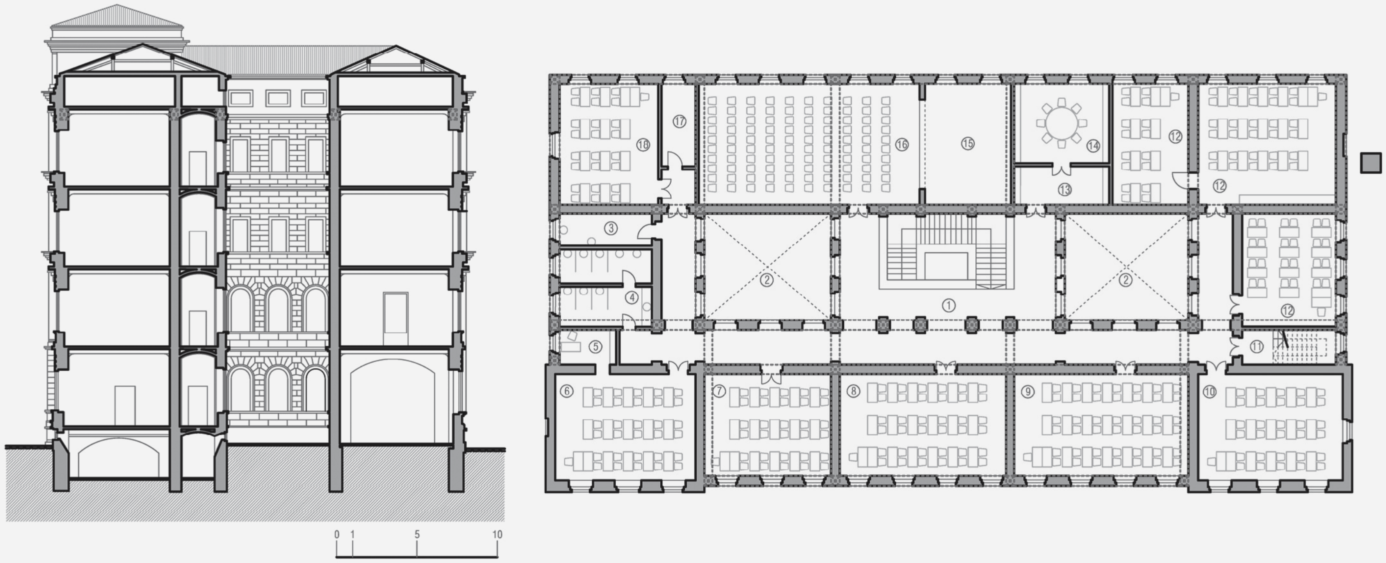


Fig. 8 Strengthening of the third floor of the I Gymnasium with horizontal bond beams (author's contribution)

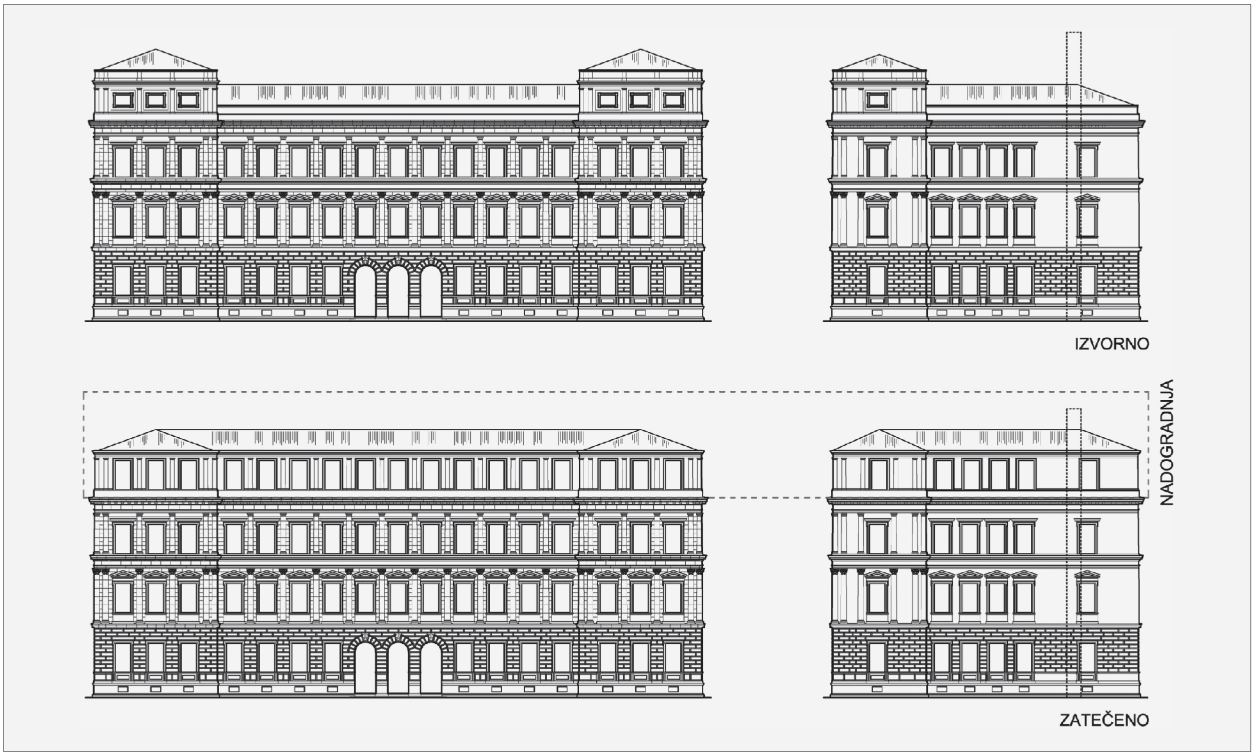


Fig. 9 I Gymnasium, construction of the third floor (author's contribution)

where magenta colour is present are of particular significance, as it indicates not only the lowest surface temperatures but also the presence of moisture in the material, considering that water, due to its thermal conductivity and evaporation, often causes characteristic thermal deviations.

The magenta colour is seen on the ground surface next to the school (see Fig. 7). This phenomenon directly indicates the capillary rise of moisture from the foundation soil towards the walls. Such moisture distribution is characteristic of masonry buildings without adequate horizontal waterproofing, where porous materials enable capillary transport of water to higher zones of the structure. Identification of these cold and damp areas utilizing the infrared thermography method is an important step in diagnosing the condition of the building, as it allows the identification of locations with an increased risk of material degradation, reduced thermal efficiency, and the development of biological damage.

It should be noted that no quantitative measurements, such as in-situ moisture content or laboratory testing of material properties, were performed as part of this study. Therefore, the interpretation remains qualitative and is based on visual assessment, historical documentation, and standard thermographic analysis. Despite this limitation, the observations provide important guidance for conservation planning, highlighting areas of concern that may require targeted monitoring, further material testing, or preventive intervention to ensure the building's structural and architectural integrity.

The construction of additional storeys of existing structures, made as unreinforced masonry structures (URM) without vertical and horizontal tie-beams, represents a significant challenge from the aspect of statics and conservation requirements, illustrated by the example of the I Gymnasium. From the perspective of seismic safety, partial and localized strengthening can cause non-uniformity in stiffness and strength distributions between the strengthened and original floors. Such discontinuities in the structural system can lead to stress concentration at transitions, which increases the building's susceptibility to seismic damage. For the strengthening measure to be effective, it is of the utmost importance to apply consistent measures to the entire height of the structure, and not just localized as is done in this structure. In this case, the original load paths were not respected, which, as a result, has an impact on the overall action of the structure once exposed to ground shaking. The construction of the third floor included the introduction of the horizontal bond beams and tie columns (marked in red), as illustrated in Fig. 8 and Fig. 10 by red colour.

Cement mortar was most probably used in the construction of the new additional floor.

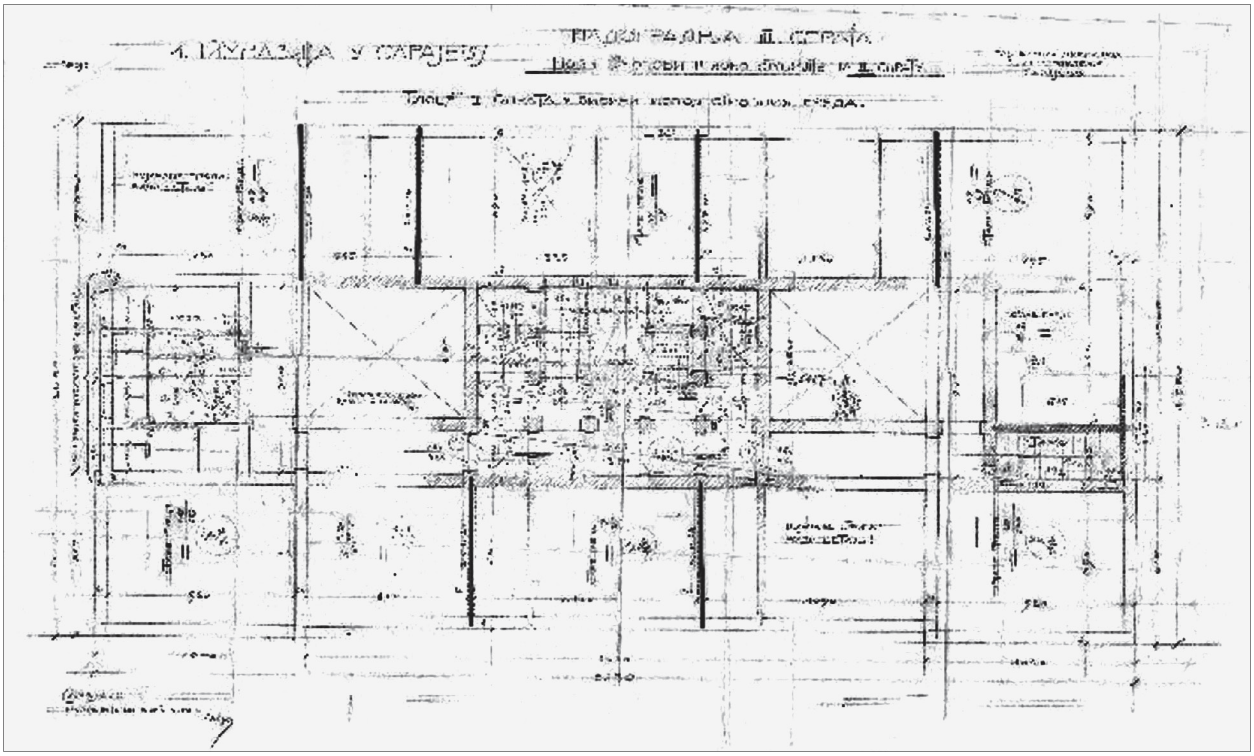


Fig. 10 I Gymnasium, structural reinforcement of the second-floor ceiling (source: archival materials of the First Gymnasium)

As the rest of the structure is made of lime mortar, this is completely inconsistent with international guidelines for historic structures, (ICOMOS, 2003: 5), where the importance of material compatibility is emphasized. Cement mortars are significantly stiffer and less vapor permeable than traditional lime mortars, which leads to mechanical and chemical incompatibility with original materials. This kind of incompatibility can have negative impacts on the structure, which can be seen in the accumulation of stress, possible crack formations, and salt accumulation. This all could and most probably would accelerate the deterioration of the structure.

Fig. 9 illustrates the current state of the structure, indicating the creation of the additional third floor. This addition has increased the mass of the structure, and with the usage of the reinforced concrete bond beams and tie columns, this has significantly altered the dynamic action of the original URM structure. The addition of the mass and the change in the stiffness of the upper floor have had an impact on the natural frequency. If the connection of the old part of the structure and the new part is not conducted in a proper way, this could have an impact on the structure's seismic response. At the location of intervention, there is an obvious stiffness and mass discontinuity, which may lead to differential movements, cracks, and stress concentrations, as seen in similar cases in Italy (Fig. 2). The dynamic properties of reinforced concrete elements (bond beams and tie columns) are not fully compatible with URM walls, together with uneven stiffness distribution, may cause the amplification of the vibration effects, especially in the transition zones, reducing overall seismic resistance.

In conclusion, the construction of the new floor added new mass to the structure, and localized reinforced concrete elements changed the localized stiffness of the structure, which altered the original dynamic action of the URM structure, increasing its vulnerability. One of the most important elements in the process of strengthening existing structures is to maintain and ensure uniform mass and stiffness distribution along the building height. This is crucial as, in this way, the dynamic behavior of the structure will not be changed, or this change will be kept to a minimum, leading to higher structural resistance.

PREVENTION AND MITIGATION OF DEGRADATION

Poor maintenance can be identified as one of the crucial causes of masonry structures' degradation (Fig. 11). If regular inspections and timely repairs are conducted, many of the causes (water infiltration, crack formation, and biological influences) can be significantly mitigated. Progressive deterioration is

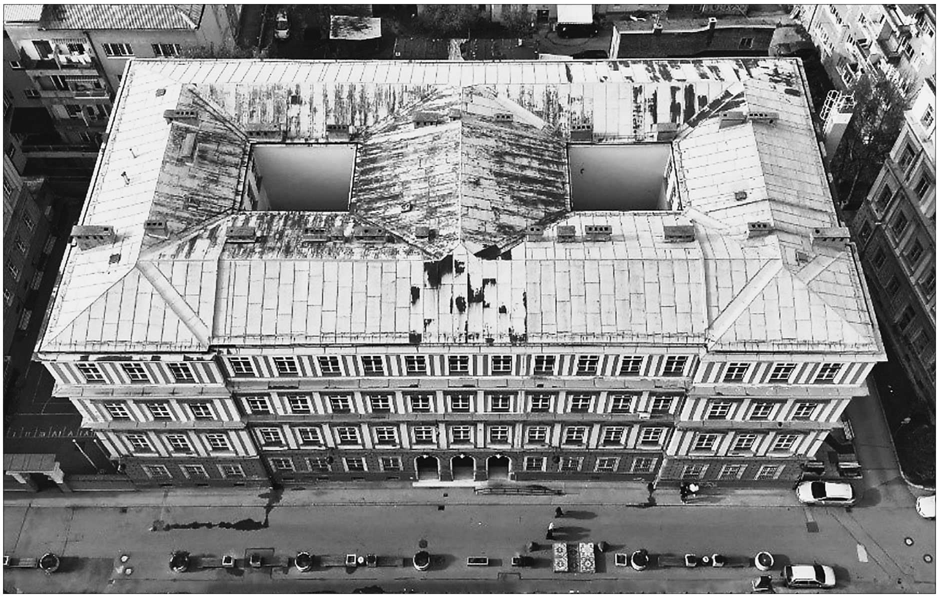


Fig. 11 I Gymnasium, inadequate maintenance (author's contribution)

usually seen on masonry structures that are not timely and well-maintained, which may lead to severe damage that requires expensive and extensive interventions.

Therefore, regular maintenance is seen as the first step for preventing the various effects that can have a direct impact on the deterioration and degradation of the material, which would lead to the reduction of the structural resistance. Key measures include proper drainage and waterproofing, removal of surface salts, timely repair of cracks, and the use of aggregates, mortars, and protective coatings that limit moisture and pollutant penetration.

Conclusion

The assessment based on the RISK-UE methodology was conducted on a I Gymnasium, a school masonry building located in Sarajevo (Bosnia and Herzegovina).

It was noted that the masonry structure exhibits low vulnerability under very weak seismic events (mean damage grade <0.5 at intensity IV), but a pronounced increase at intensities VII–VIII, characteristic of Sarajevo, where mean damage grades reach 2.12 and 3.19.

At intensity VII, the structure is most likely to experience moderate (≈34.3%) and substantial damage (≈25.3%), while at intensity VIII, the probability of substantial damage increases to ≈34%, and very serious damage affects approximately 30% of cases, confirming a non-linear escalation of vulnerability. These results are consistent with observed damage patterns in comparable school buildings of similar typology in Croatia, providing validation of the applied methodology. The analysis's primary drawback is that it ignores the stronger floor; a more thorough evaluation that takes this factor into consideration will be carried out in later assessments.

Infrared thermography and drone usage proved to be very helpful tools, as it was possible to determine the current condition of the building's envelope. With the help of infrared thermography, defects of the structure were identified, such as the location of cracks, moisture regions, and other signs of degradation. The findings further highlight that material degradation, and non-uniform strengthening interventions significantly reduce seismic resilience, emphasizing the need for systematic maintenance and comprehensive, compatible retrofitting strategies.

Regular inspection, together with proper and timely maintenance, can lead to the preservation of cultural heritage buildings and the extension of their service life.

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