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SEISMIC ASSESSMENT OF A HISTORIC AUSTRO-HUNGARIAN MASONRY BUILDING IN SARAJEVO'S BAŠČARŠIJA

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Abstract

The main purpose of this paper is to assess the seismic resistance of a masonry building from the Austro-Hungarian period in Sarajevo. The building is situated in Sarajevo's old Baščaršija, which is well-known for its marketplace of tiny adobe and wooden buildings from the Ottoman era. It is characterized by specific Austro-Hungarian architecture from the rebuilt Latin district next to the Miljacka River. European construction rules were created following a fire in 1879. The structure is notable for its size and design, which combines Ottoman surroundings with Austro-Hungarian influences. The original structure had two floors A business area occupied the ground floor, while residential apartments occupied the top floor of the original building, which was recorded by the Governmental Building Department in 1903. It was a typical residential rental building at the time. Later, a second level was constructed while keeping the same layout and structural elements. Typical Austro-Hungarian solid bricks from that era were used to construct the load-bearing walls, with lime mortar for the joints. Sand infill serves as fire-resistant insulation between the wooden beams and boards that form the floor structure. The original pitched roof was made of wood.

Numerical modeling and nonlinear static (*pushover*) analysis were conducted using the 3Muri software package. The 3Muri software package, specialized in analyzing masonry structures, employs the innovative Frame by Macro Element (FME) method, enabling detailed seismic behavior analysis of walls. This paper presents detailed pushover analysis results, covering the distribution of lateral forces (uniform and static) for horizontal acceleration in the X and Y directions, considering the significant damage state for a 475-year return period. The main parameter monitored during the analysis was the vulnerability indexes.

Results are presented for all walls, and wall damage was analyzed relative to the direction of seismic action, identifying walls most affected by bending or shear forces.

Keywords: masonry; pushover analysis; Austro-Hungarian buildings, timber floor, 3Muri software, historical structures

1. Introduction

Recently, frequent and intense ground shaking has occurred in the Balkan region. In Bosnia and Herzegovina (BiH), significant tremors were felt following the earthquake in Croatia in 2020. The epicenter of this earthquake was located in Petrinja, with a magnitude of 6.7 on the Richter scale. In November 2019, Albania was struck by an earthquake with a recorded magnitude of 6.4. More recently, in April 2022, BiH experienced an earthquake with a magnitude of 5.7 on the Richter scale, with its epicenter near Stolac. According to the Federal Hydrometeorological Institute, this earthquake was considered stronger than the one that occurred in 1969 in Banja Luka [1].
The initial phase of this research involved field investigations of 700 buildings (Fig. 1a) which provided the necessary parameters to establish a database for calculating the seismic vulnerability of buildings under earthquakes of varying intensities [2]. Seismic analysis was conducted for a residential building located at Aščiluk Street (Fig.1b), as it was included in the field study and was identified by Ademović et al. [2] as the most vulnerable among the buildings in its neighborhood. This building was constructed in 1903 during the Austro-Hungarian period, in accordance with the regulations enforced at that time.

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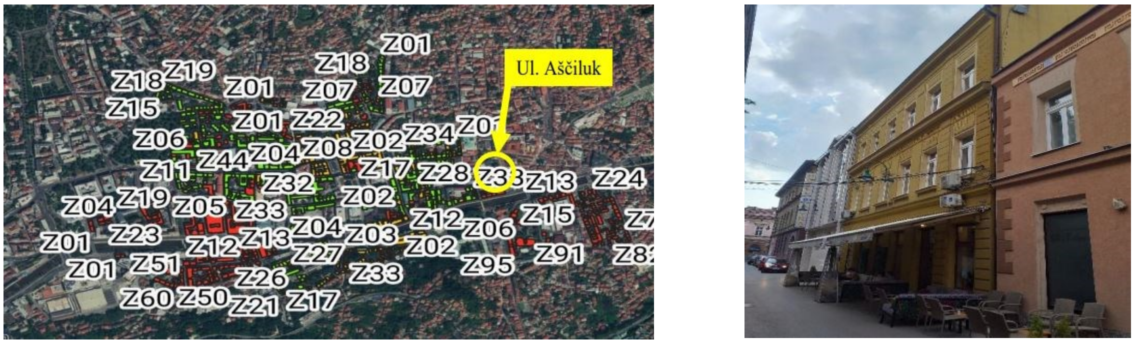


a) b)

Figure 1. a) Location of the analyzed building; b) Building in the Aščiluk Street [2].

2. Building Description

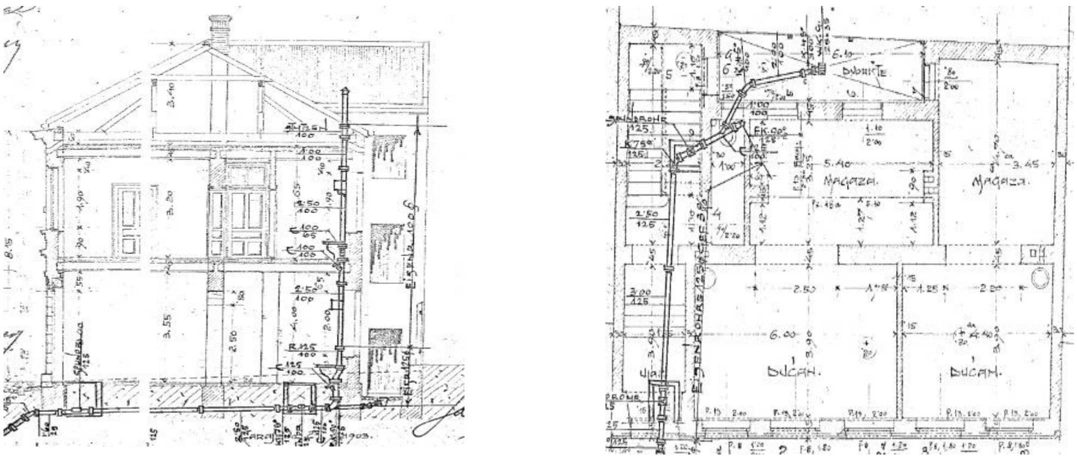
2.1. Structural Geometry and Material Specifications of Load-bearing walls

The building represents a typical unreinforced masonry structure (URM) with wooden floor structures, characteristic of the time it was constructed. The building consists of a ground floor, one upper floor, and an attic. The original blueprints were obtained from the Institute for Protection of Cultural-Historical and Natural Heritage of Canton Sarajevo and are presented in Fig. 2. As per the original site plan, the building was designed as an infill structure between two adjacent buildings. Its longer side, measuring 13.30 meters, is aligned in a north-south orientation, with the façade facing the street situated to the south, the shorter side is 10.40 m and has a total height of 10.15 m.

Two external load-bearing walls, positioned longitudinally and transversely, make up the building's structural system. The longitudinal load-bearing walls govern the layout of both floors. Chimneys are part of a central longitudinal load-bearing wall that is 4.35 meters from the façade that faces the street. A transverse load-bearing wall on the ground floor is situated 2.15 meters away from the outside transverse wall on the west side. The first 0.95 meters of this 4.80-meter-long wall are 45 cm thick, whereas the following portion is only 30 cm thick.

The first and second levels have partition walls built above this wall. At a distance of 2.50 meters from the center wall, another transverse wall varies in thickness from 30 cm over 3.70 meters to 45 cm over 1.85 meters. The transverse wall and the staircases run through the two higher stories and are situated in the northwest corner. The arrangement of the 15 cm thick partition walls is depicted in Fig. 2.

Sand infill serves as fire-resistant insulation between the wooden beams and boards that make up the floor structure. For fire safety considerations, the structural floor was separated from the lower level, and the original pitched roof was constructed of wood.



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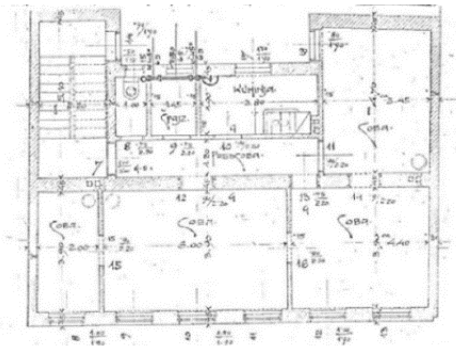

Figure 2. The building's original designs, including the cross-section, ground floor, and first-floor layout (Source: The Cantonal Institute for the Protection of Cultural, Historical and Natural Heritage, Sarajevo)

The structural system from the Austro-Hungarian period is based on longitudinal/medium load-bearing walls, the thickness of which depends on the regulations of the *Construction Code*. The walls of the lower floors, primarily made of brick and unreinforced, can be up to one meter thick [3]. Fig. 3 shows the walls around the elevator in the basement, while those on the upper floors are reduced by 15 cm. According to the *Construction Cod*e, the thickness of the load-bearing walls on the top floor must be at least 45 cm, increasing by 15 cm on each lower floor. The examined building has two floors, with the middle and outer load-bearing walls remaining 45 cm thick. The arrangement of load-bearing walls in only one direction may lead to structural failure due to an earthquake, as illustrated by the example of the 1963 Skopje earthquake [4].

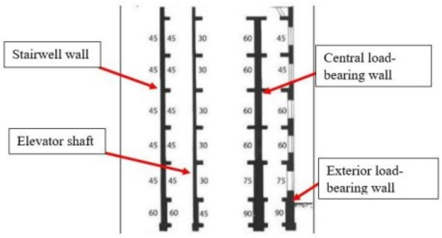

Figure 3. Thickness of the walls according to the Construction Code [5].

2.2.Structural Geometry and Material Specifications of the Timber Slab

The floor-ceiling assemblies characteristic of the Austro-Hungarian period are constructed of flexible wooden systems, with spans ranging between 4 m and 6 m and thicknesses varying from 45 cm to 55 cm. The floor structure layout of the building is depicted in Figure 4 and has been modeled using the 3Muri software for advanced seismic analysis. Fig. 4 shows the orientation of the timber joists (20x26cm) located between the boards. These transversely oriented joists are supported by the central longitudinal wall and the façade walls.

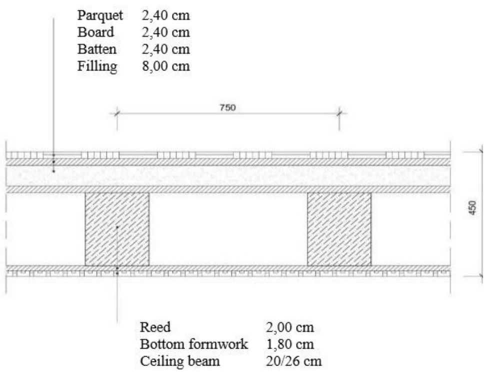

Figure 4. Cross-section of the timber floors [6].

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Wooden ceilings are the oldest and most widespread type of ceiling until the introduction of reinforced concrete in the 21st century. Flexible ceilings, typically made as beams supported by walls, distribute lateral seismic forces to vertical load-bearing elements based on the corresponding surface area. Unlike rigid ceilings, flexible ones are not capable of redistributing forces caused by torsional deformations of the structure.

The consideration of the stiffness or flexibility of ceilings aims to determine the way seismic forces are distributed to load-bearing elements (walls), and whether the forces are distributed based on the stiffness of the walls or the surface area. In the case of rigid ceilings, the load is transferred proportionally to the stiffness of the vertical elements, while in flexible ceilings, the load is transferred according to the surface area. The distribution of seismic forces depends on the stiffness of the ceilings and the vertical load-bearing elements. According to El Sherbeny [7], most ceilings have characteristics that are between rigid and flexible.

Flexible ceilings can cause torsional effects due to the varying stiffness of the vertical load-bearing elements, but they are not stiff enough to transfer forces to walls perpendicular to the seismic movement direction, which can lead to structural failure. On the other hand, rigid ceilings distribute torsional forces to vertical elements in proportion to their stiffness. Torsional effects can cause damage at different levels, making it essential to pay attention to the type of ceiling, especially in irregularly shaped structures. As shown in Fig. 5, with rigid ceilings, each wall takes on a portion of the force proportional to its stiffness relative to the total stiffness of the structure, while with flexible ceilings, the force distribution follows the surface area, so the middle wall takes on twice the force share (V/2) compared to the outer walls (V/4).

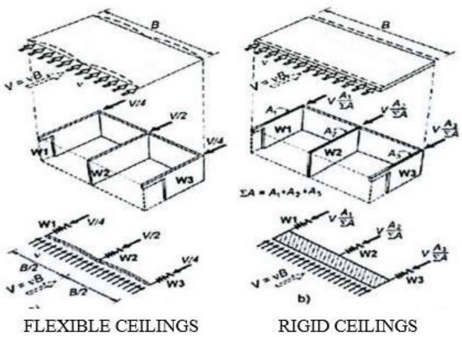

Figure 5. Distribution of seismic forces to vertical load-bearing elements (walls) [7].

Based on observed damage from strong earthquakes, insufficient structural integrity, and weak connections between vertical walls, and between the walls and ceiling structures can cause the separation of load-bearing walls at corners and joints, leading to the collapse of gable or vertical walls in the direction of seismic ground movement [4]. Experimental studies [8, 9] show that ceiling structures with beams supported only by walls cannot prevent wall separation, which may lead to a partial collapse of upper floors (Fig. 6a). However, connecting the walls with steel ties ensures structural integrity and fully utilizes the resistance of the walls (Fig. 6b).

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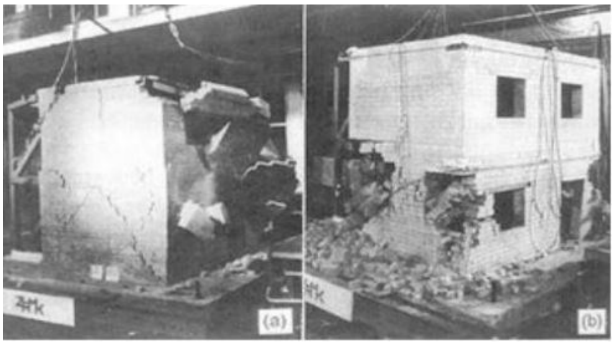

Figure 6. Failure mechanism of the masonry building models made of brick: a) Without wall ties; b) With wall ties [4].

Table 1. Material parameters for masonry [6]

Characteristics-masonry	Symbol	Value
Compressive strength	f_c	3.9 N/mm^2
Tensile strength	f_t	0.02 N/mm^2
Youngs modulus	E	2730 N/mm^2
Shear modulus	G	910 N/mm^2
Mass density	ρ	13.44 kN/mm^3

Table 2. Material parameters for timber [6]

Characteristics-timber	Symbol	Value
Youngs modulus	E	7000 N/mm^2
Shear modulus	G	440 N/mm^2

3. Numerical model

3.1. Pushover analysis

Pushover analysis is conducted under a constant gravitational load combined with an incrementally increasing horizontal load that simulates seismic effects. Horizontal forces are applied at floor levels, where masses are concentrated, following two primary distributions: a uniform distribution along the building height or a modal distribution corresponding to the deformation shape of specific natural vibration modes. This method tracks the structural response to a monotonically increasing static horizontal load until structural instability is reached. It is particularly suited for assessing existing buildings and incorporates nonlinear material behavior.

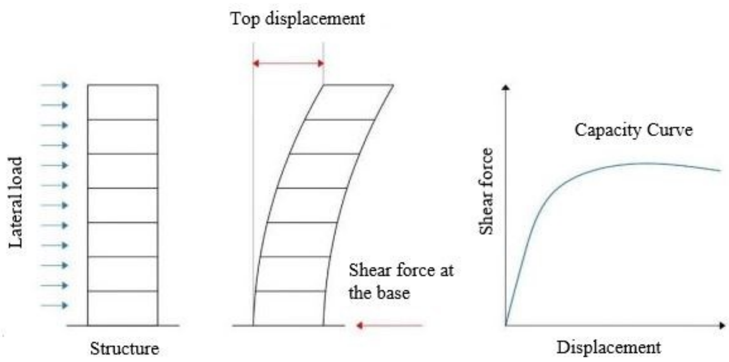

Figure 7. Capacity Curve [6].

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The process involves applying incrementally increasing forces while monitoring the structural response. Given the nonlinear nature of the analysis, appropriate nonlinear material behavior is assumed. The seismic response considers parts of the structure transitioning into nonlinear behavior, which is crucial for evaluating load-bearing systems. The analysis tracks the displacement of the top floor relative to the applied horizontal force, serving as a reference. The result is an F-Δ diagram, or capacity curve, which illustrates the relationship between the total lateral force and the building's top displacement (Fig.7).

3.2. Numerical modeling using 3Muri software

Seismic analysis was conducted using the 3Muri software package, which specializes in the analysis of masonry structures. 3Muri employs the Frame by Macro Element (FME) method, specifically designed for masonry structure analysis. After modeling the walls and floor diaphragms, the software automatically generates a mesh. The innovative FME method schematically represents the structure as an equivalent frame composed of macro elements. This approach facilitates a deeper understanding of seismic behavior and provides the designer with essential information for a detailed review of the results.

The equivalent frames offer a comprehensive mechanism for analyzing in-plane damage, including shear and combined axial loading, while out-of-plane wall collapse is not considered. Due to their simplicity and satisfactory accuracy, equivalent frames are considered the most suitable tool for relatively quick assessments of the seismic resistance of masonry structures.

According to the equivalent frame method, the numerical model consists of vertical load-bearing elements ("piers") and horizontal elements ("spandrels"), which are connected by rigid nodes ("rigid portions") as illustrated in Fig .8 ([10], [11]).

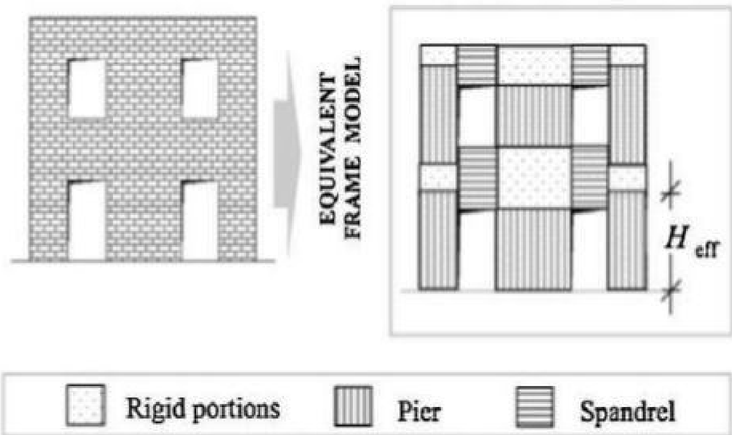


Figure 8. Idealization of a wall using the Equivalent Frame Method [11].

The 3D model of the analyzed structure is presented in Fig 9. The 3Muri software package operates on the principle of simplified numerical models for the analysis of masonry structures, where equivalent frames are formed before the actual calculation. In this method, the numerical model consists of vertical load-bearing elements and spanning elements connected by nodes, as shown in Fig. 9.

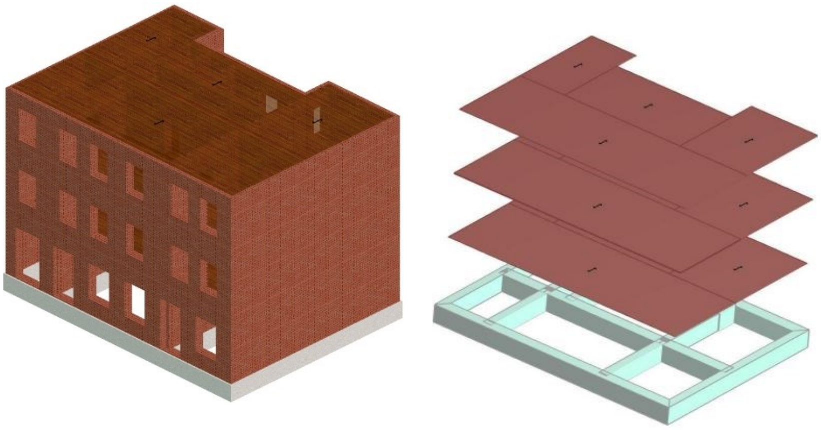
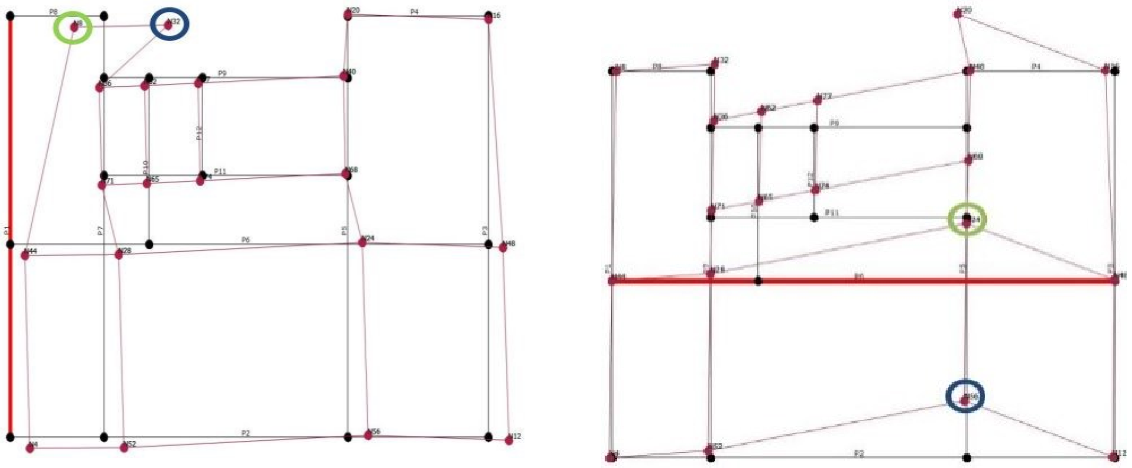


Figure 9. 3D model of the structure [6].

A static analysis was performed to verify the structure's capacity to withstand vertical loads, and the results were satisfactory.

Before conducting the pushover analysis, a modal analysis is performed to determine the fundamental dynamic characteristics of the structure, such as its natural frequencies and mode shapes. Based on the mode shapes, i.e., the deformation patterns of the structure, control points are selected for both directions (X and Y) on the highest floor of the building, as shown in Figs. 10a and 10b.



a) b)

Figure 10. Modal analysis: a) Control displacement in X direction; b) Control displacement in Y direction.

Table 3 displays the results from the modal analysis, including the natural periods and mass participation coefficients for the relevant directions. The first mode (shown in green) occurs in the Y direction and activates 35.82% of the mass, while the second mode (shown in red) occurs in the X direction, activating 43.78% of the mass.

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Table 3. Modal analysis results.

Active in pushover X dr.	Active in pushover Y dr.	Mode	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
☐	☐	1	0,28842	39.230,51	11.09	0.64	0.00	0.13	0.00
☐	☐	2	0,19648	194,46	0,05	126.730,54	35,82	2,19	0,00
☐	☐	3	0,17448	2.135,09	0,60	54.384,92	15,37	7,10	0,00
☐	☐	4	0,14122	26.957,38	7,62	5.264,41	1,49	2,11	0,00
☐	☐	5	0,13819	154.894,53	43,78	10.152,23	2,87	35,97	0,01
☐	☐	6	0,12775	46.396,23	13,11	3.043,70	0,86	3,31	0,00
☐	☐	7	0,11889	6.252,65	1,77	24.377,84	6,89	20,54	0,01
☐	☐	8	0,10682	41.553,43	11,74	3.058,91	0,86	2,47	0,00
☐	☐	9	0,09198	8.909,66	2,52	29.796,31	8,42	5,16	0,00
☐	☐	10	0,08451	310,50	0,09	32.419,18	9,16	31,12	0,01
☐	☐	11	0,07745	595,95	0,17	3.454,62	0,98	31,25	0,01
☐	☐	12	0,07323	394,41	0,11	18.182,98	5,14	4,60	0,00

The pushover analysis was conducted for Node 8 in the X direction, as it exhibited the greatest displacement response. Additionally, considering the significance of the middle longitudinal bearing wall in Austro-Hungarian construction, especially for buildings of this period [3], Node 24, located on the longitudinal bearing wall (P6), was also selected for analysis. The pushover analysis was performed for Node 24 in both horizontal directions, resulting in a total of 24 distinct calculations. The analysis was carried out across four lateral force application directions (+X, -X, +Y, -Y), with two distinct lateral force distributions (uniform and static), and with or without seismic force eccentricity, which could be either positive or negative.

4. Results of numerical analysis

The distribution of transverse forces was analyzed under both uniform and static force distributions, with calculations performed for horizontal accelerations in the X and Y directions. The analysis was carried out for two limit states: limited damage and significant damage, corresponding to return periods of 95 and 475 years, respectively. The results are presented for various structural elements, including the middle longitudinal bearing wall (45 cm thickness), the street-facing wall (45 cm thickness), external and internal walls (30 cm thickness), and partition walls (15 cm thickness).

To evaluate the limit state of significant damage, as per Eurocode 6 [12], the condition (1) is applied:

$$d_t^{SD} \leq d_m^{Sd} \tag{1}$$

In the pushover analysis, the key parameter is the damage index, denoted as α (seismic vulnerability), which is calculated for three limit states: Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). For the Significant Damage (SD) state, the displacement is taken from the capacity curve at the point where the horizontal force drops below 80% of the maximum force. For the Damage Limitation (DL) state, the displacement corresponds to the point where the first plastic hinges form and the structure transitions into the plastic regime.

The damage index α is calculated for each limit state as indicated in equation (2)

$$\alpha_{SD} = \frac{PGA_{CSD}}{PGA_{DSD}} \tag{2}$$

where PGA_{CSD} represents the peak ground acceleration at significant damage, and PGA_{DSD} refers to the spectral peak ground acceleration at significant damage. The damage index α represents the ratio of the ultimate displacement $d_m$to the target displacement d_t for each limit state. If this ratio exceeds 1.0, the limit state is considered to be satisfied.

For this analysis, the significant damage limit state (SD) is considered the governing factor, while the limited damage (DL) state is not relevant. As a result, for all 24 combinations tested, the conditions defined by Eurocode 8 are not met, and the cells representing the DL state are marked in red.

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4.1.Comparison of Static and Uniform Force Distribution for the X-Direction

When evaluating load-bearing capacity, ultimate displacement at the Significant Damage (SD) limit state, the shape of the capacity curve, and structural ductility during the analyses with static and uniform force distributions, no significant differences are observed, as indicated by the capacity curves shown in Fig. 11. However, disparities emerge in the damage sustained by specific walls, particularly the external 30 cm thick wall in the Y direction, as presented in Fig. 12. In the static force distribution analysis, this wall remains undamaged, whereas, in the uniform force distribution analysis, it experiences initial bending damage at the ultimate displacement. Conversely, the street-facing and central longitudinal load-bearing walls exhibit nearly identical behavior in both analyses, as noted in Fig. 12.

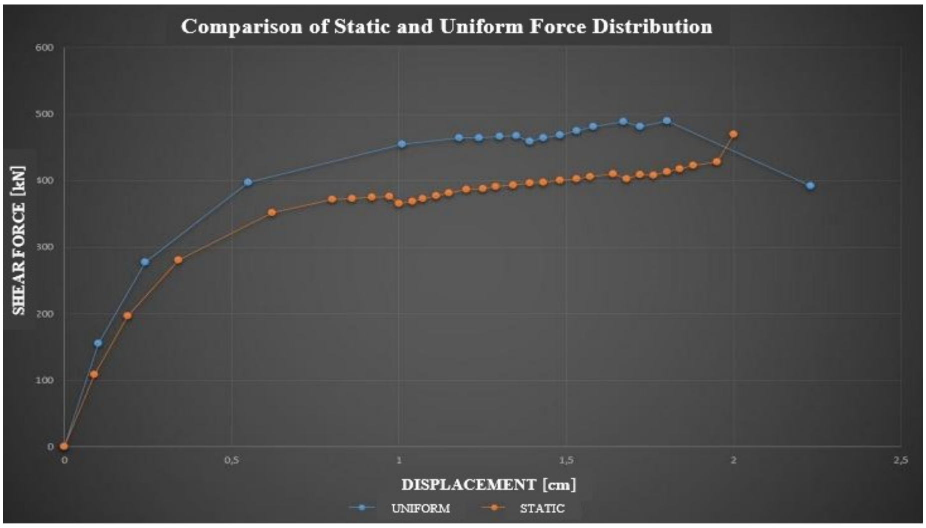

Figure 11. Comparison of Static and Uniform Force Distribution for the X Direction.

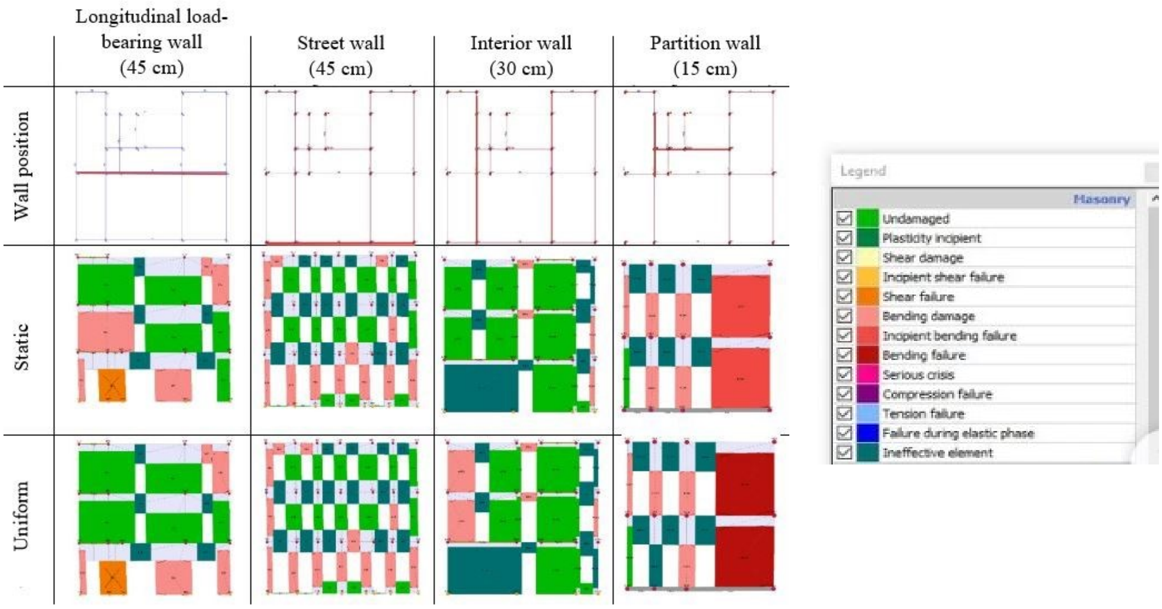

Figure 12. Damage to Walls Depending on Force Distribution – X Direction.

4.2.Representation of Wall Damage in the Analyzed Structure Relative to the Direction of Action

Fig. 13 presents the observed damage to the structural walls under horizontal loading in both the X and Y directions. The analysis indicates that under horizontal action in the X direction, the central longitudinal load-bearing wall experiences bending failure, while in the Y direction, this wall remains

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relatively undamaged. The street-facing wall exhibits initial bending cracks under horizontal loading in both directions, with similar damage patterns observed in each case. The external walls (30 cm thick) remain undamaged under horizontal loading in both directions.

Internal walls show no damage under horizontal action in the X direction; however, under horizontal loading in the Y direction, bending cracks and failures are evident, along with shear failures in certain elements. The backyard load-bearing wall sustains bending failures on the first and second floors under horizontal loading in the Y direction but remains undamaged under the X direction. Partition walls incur bending failures under horizontal loading in both directions.

Based on these observed damage patterns, the subsequent section will discuss potential approaches for repair and strengthening of the affected structural elements.

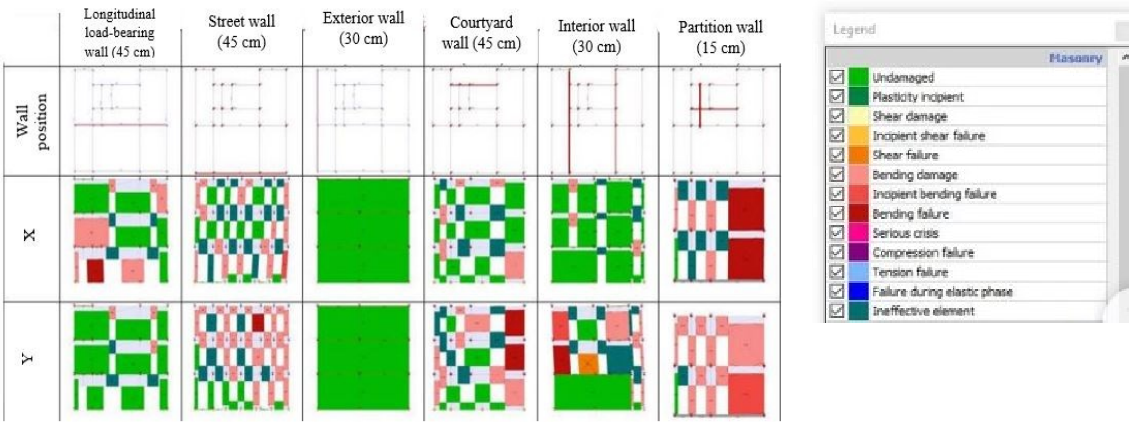

Figure 13. Representation of Wall damage relative to the direction of action.

5. Conclusions

The pushover analysis revealed that the primary damage to the building occurs due to bending and shear, irrespective of the direction of horizontal loading. A key parameter monitored during this analysis was the damage index (α), which represents the ratio of the ultimate displacement (dm) to the target displacement (dt) for each limit state. The comparison between uniform and static load distribution analyses indicated that the static load distribution provides greater structural capacity. Specifically, in the X direction, the capacity is approximately 500 kN with $\alpha = 0.743$, while in the Y direction, the capacity is about 400 kN with $\alpha = 0.733$.

Under applied loads in the X direction, the central longitudinal load-bearing wall (45 cm thick) exhibited bending damage, with failure occurring in one element. In contrast, no significant damage was observed in the Y direction. The pushover curves further indicated that the structure behaves more ductile under horizontal action in the Y direction.

A characteristic feature of Austro-Hungarian buildings is the presence of numerous openings in the walls, which often lead to significant damage. This was evident in the observed damage to this structure. Predictably, partition walls (15 cm thick) suffered bending failures across all floors, with some damage occurring at the early stages of horizontal load application. This confirms that partition walls bear part of the seismic load and should not be overlooked in seismic analysis.

Considering that the building was constructed in 1903, without explicit seismic design considerations, it is not surprising that some analyses did not meet the criteria for significant and limited damage limit states. Many Austro-Hungarian buildings in Sarajevo hold cultural, historical, architectural, and aesthetic value. Strengthening these structures must be approached cautiously to preserve their authenticity and significance. Following the 2020 Zagreb and Petrinja earthquakes, which caused significant damage to similar buildings, it is crucial to implement appropriate repair and strengthening measures to enhance seismic resistance.

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