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ENGINEERING ANALYSIS OF A TYPICAL PRE-CODE URM BUILDING IN ZAGREB

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Abstract

This paper evaluates the seismic resistance and vulnerability of an existing unreinforced masonry (URM) building in Zagreb's Folnegovićevo neighborhood. The building with potentially low seismic resistance was erected in the 60s, a period characterized by the introduction of the first seismic code in practice. It has a rectangular plan 36.9 m x 10.7 m and consists of a basement, a ground floor and four floors totaling 17 m in height. The solid brick walls with variable thickness are evenly distributed in both directions and they run continuously from the foundation to the top. The reinforced concrete slabs are 14 cm thick rigid diaphragms made of MB 20 concrete class with smooth rebars GA 240/360.Two structural variants were analyzed: one with reinforced tie-beams and one without. Equivalent Static Analysis was conducted manually and, a Time-History Linear Dynamic Analysis was performed with the Tower Radimpex software for preliminary assessment. This method provided a basic assessment of the building's seismic resistance, while also comparing manual and numerical results. Two advanced methods, Nonlinear Time-History Analysis (NLTH) and Pushover Analysis, were conducted with Diana FEA software employing Engineering masonry constitutive law to describe the highly nonlinear behaviour of masonry walls which can crush, crack, or fail in shear. NLTH simulated the building's response to the 2020 earthquake, identifying critical structural components and potential damage. The Pushover Analysis evaluated how the building responded to increasing lateral loads, focusing on seismic resilience and collapse mechanisms. This research provides an in-depth comparison of different analytical approaches, contributing to the methodology for assessing and strengthening existing buildings. Conclusions drawn from these analyses emphasize recommendations for modifications to improve structural performance and safety.

Keywords: pre-code URM building, macro-modeling, equivalent static analysis, NLTH

1. Introduction

The city of Zagreb, Croatia, experienced a strong earthquake on March 22, 2020, with an initial magnitude of 5.5 on the Richter scale. Within a little over 24 hours after the initial earthquake, the region of Zagreb recorded a total of 57 additional seismic events. These ground tremors resulted in extensive and significant damage to existing masonry structures in Zagreb. In order to assess earthquake response, static and dynamic analyses are conducted for an existing residential building located in the Folnegovićevo neighborhood in Zagreb, selected as a structure with potentially low seismic resistance. The analysis of the current state of the residential building was based on a review of documentation from the State Archive in Zagreb. This review revealed that, according to the development program for the Folnegovićevo, Trnsko, and Selska Street neighborhoods, standard designs for four-story blocks with low and high ground floors were created in 1960. Three standard units were designed—types A, B, and C—which allowed for the combination of multiple units.

As a consequence of the catastrophic earthquake in Skopje in 1963, the 1964 Provisional Technical Regulations for Construction in Seismic Areas were introduced. These regulations brought the first national seismic map with seismic zones ranging from VI to IX based on the Mercalli-Cancani-Sieberg scale, derived from the 1950 version. According to this map, the residential building is located in the IX seismic zone [1].

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Residential buildings of the Bartolić type were designed in 1960 and were not planned in accordance with the Regulation on Provisional Technical Standards for Construction in Seismic Areas. A review of documentation from the State Archives in Zagreb revealed that most of these buildings were under construction at the time the new regulations were adopted. In 1964, a commission for the technical inspection of buildings was established. A series of structural measures aimed at strengthening residential buildings against earthquake forces were introduced, focusing on the construction of vertical reinforced concrete ties (columns and ring beams) and the connection of load-bearing walls with horizontal reinforced concrete ring beams.

Based on the documentation from the State Archives in Zagreb, it was determined that, as a result of these structural measures, some buildings were constructed with ring beams, while others were not. For this reason, this study will analyze a building in which, in one case, the vertical load-bearing system for resisting seismic forces consists of masonry without ring beams, and in the other case, masonry bounded by ring beams.

1.1.Building description

The building was designed in 1960 and consists of a basement, ground floor, and four upper floors. The residential building has a rectangular floor plan with maximum dimensions of 36.9 m x 10.7 m, and the total floor area of one story is 394.8 m². The height of the basement is 2.56 m, while the height of the other floors is 2.66 m each. The total height of the building is 17 m. The roof is flat and designed as an inaccessible terrace with rolled gravel, thermal insulation, and water proofing.

a)

b)

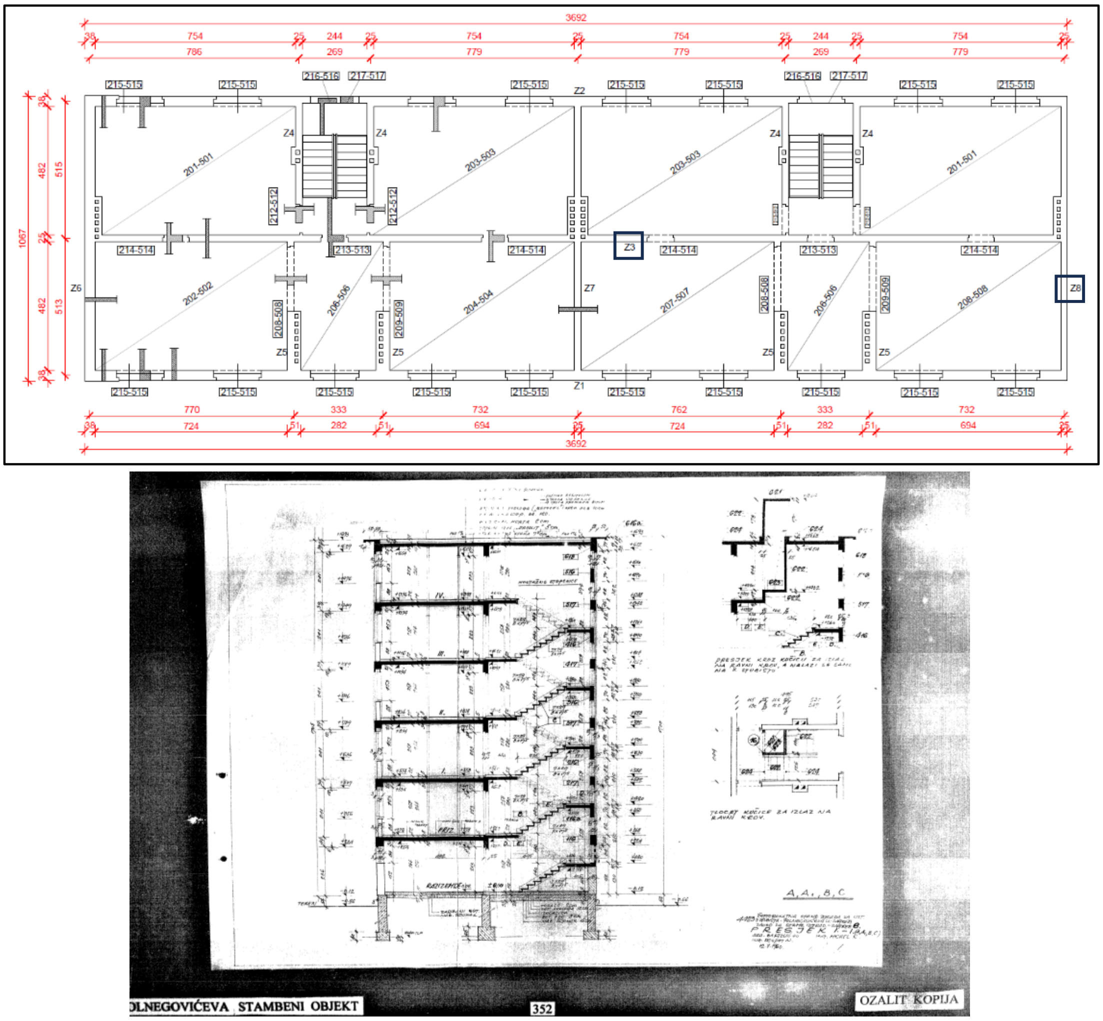

Figure 1. a) Building plan; b) Cross – section (State Archive in Zagreb).

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The structure consists of interconnected solid brick walls evenly distributed in both directions. The exterior load-bearing walls in the basement are 38 cm thick and made of brick, while the central wall is 25 cm thick and made of concrete (MB-110). Load-bearing walls in the ground floor and upper floors are made of brick, with brick grades and mortars specified for each floor according to the structural calculations. One external load-bearing wall is 25 cm thick (in the shorter direction), while the other external load-bearing walls are 38 cm thick. The thickness of the internal load-bearing walls is 25 cm. Based on the test data for solid bricks conducted by the IGH Institute Zagreb, it was determined that the bricks correspond to grade M15. The project specifies that the mortar should have a volumetric ratio of 1:2:6 = cement:lime:sand, which approximately corresponds to mortar grade M5. If no experimental data are available, the characteristic compressive strength of masonry constructed with general-purpose mortar can be determined using empirical formulas.

All load-bearing walls run continuously from the foundation to the top of the building, ensuring that the structure meets vertical regularity criteria. The share of the surface area of lateral load-bearing walls in the total gross floor area of the building's floor plan for the X direction is 5.58 % (4.96 % in Y).

Partition walls are made of hollow Troskanit blocks with a thickness of 15 cm. The staircase wall is 25 cm thick. Window lintels in the basement and upper floors are monolithic, while door lintels are prefabricated. The stairs are prefabricated, supported by beams in the stair landing, which is the same height as the landing.

The floor structure is constructed as a 14 cm thick reinforced concrete slab, which can be considered an axially rigid diaphragm, meaning deformations at the ceiling level can be neglected. The structure is made of concrete (MB 20) and reinforcement steel (smooth bars GA 240/300). The floor structure is directly supported on the brick walls but is not adequately connected to the walls to allow for proper distribution of seismic forces across individual walls or to prevent separation during seismic events.

2. Modeling strategies for masonry

Masonry is a composite material that consists of units and mortar. While experimental investigations are crucial for understanding the structural behavior of masonry, numerical modeling serves as a valuable complement, offering additional insights. Masonry structures are typically analyzed using finite element methods (FEM), and computational strategies are traditionally classified into three categories based on the level of detail: micro-, meso-, and macro-modeling techniques. Another group of modeling approaches includes the discrete element method (DEM) and hybrid methods combining finite and discrete elements (FEM/DEM). The equivalent frame (macro-element) approach is specifically designed for analyzing masonry panels and piers. No single modeling strategy can be universally preferred, as each is suited to different applications.

2.1. Micro-modeling

Micro-modeling, a detailed approach to analyzing masonry, aims to represent all the constitutive elements of a masonry structure, including units, mortar, and the unit/mortar interface. In this method, mortar and brick units are discretized using continuum elements with corresponding failure criteria, while interface elements between the mortar and units introduce discontinuities in the displacement field to account for potential cracking. However, due to its complexity, micro-modeling is rarely employed.

This method is most suitable for analyzing small structural elements, particularly when there is a need to investigate strongly heterogeneous stress and strain states to gain deeper insight into local behavior [2]. The primary goal of micro-modeling is to accurately represent masonry by incorporating the properties of each constituent and their interactions. The required experimental data must be derived from laboratory tests conducted on the individual components and small masonry samples.

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2.2. Meso-modeling

In meso-modeling, wall geometry is slightly simplified by homogenizing the mortar joints and mortar-brick contact into a single discontinuous interface element, while the brick units are expanded to maintain the overall geometry. Masonry is treated as an assembly of bricks connected by potential fracture or slip lines at the joints. Additionally, an interface can be added at the midpoint of each brick to account for potential cracking.

This approach omits the Poisson effect of mortar, which generates biaxial tension in the brick units, limiting the model's ability to capture all possible failure modes [3].

2.3. Macro-modeling

Macro-modeling is suitable for structures composed of solid walls with sufficiently large dimensions, where stresses and strains can be averaged over a macro-length. In large-scale, practice-oriented analyses, the interaction between units and mortar is typically negligible for the overall structural behavior. These models approximate a heterogeneous masonry wall as a single, uniform material, with discretization independent of the brick layout—units, mortar, and the unit-mortar interface are effectively smeared into a continuum. This phenomenological approach requires material properties to be derived from tests on masonry samples of sufficient size under homogeneous stress conditions. A comprehensive macro-model must account for orthotropic behavior, incorporating different compressive and tensile strengths along material axes and distinct inelastic behaviors for each axis. However, the complexity of introducing orthotropic properties means that only a few macro-models have been developed.

Macro-modeling is practice-oriented, offering reduced computational time and memory requirements and simplified mesh generation [4,5, 6, 7]. It is particularly valuable when a balance between accuracy and efficiency is necessary. Nevertheless, capturing the complex failure mechanisms of masonry walls remains challenging, as various failure modes can occur independently or in combination. Numerical issues may arise in cases of strong localization with prominent macro-cracks. In macro-models, cracking is smeared across a characteristic length, which does not accurately reflect real conditions.

In this paper Engineering Masonry model is used for nonlinear modeling of masonry behavior in DIANA [8, 9]. The model has been proposed by DIANA FEA and the Technical University of Delft to evaluate the building masonry stock after a series of earthquakes caused by gas extraction in the Groningen area [10]. Engineering Masonry model is an orthotropic total-strain continuum model with smeared cracking and it can be used with membrane or shell elements. The model is capable of simulating compression, tensile and shear failure modes and it can crack in the X-horizontal bed joint, the Y-vertical head-joint as well as diagonally in the form of staircase. In comparison with the Total Strain Crack model [8], the Engineering Masonry model defines unloading behavior in compression more realistically by assuming combined damage-plasticity model. Coulomb friction failure criterion is included to model typical shear failure. The tensile behavior is defined by linear loading and softening curves.

2.4. Discrete modeling

When significant geometric and material nonlinearity arises in masonry structures, such as fracturing or crushing, finite element methods (FEM) can struggle to accurately account for material damage and discontinuity. To address these challenges, the discrete element method (DEM) offers an alternative approach, modeling fragmentation by representing the medium as an assembly of discrete elements [11]. The DEM model requires relatively few material properties, as the complexity of its response stems from the collective behavior of simple individual elements. While the elements themselves are generally linear elastic, their overall behavior can exhibit nonlinearity, dilation related to mean stress, transitions from brittle to ductile behavior, hysteresis, and more. Additionally, DEM inherently captures phenomena such as localization (e.g., fractures in brittle solids or shear bands in granular materials), which can pose challenges for mesh-based models. However, a key limitation of the discrete approach

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lies in determining the required material properties, such as normal and shear stiffness, damping coefficients, and interparticle bonds. These parameters can only be established through macroscopic experiments, where input values are calibrated against experimental results.

A method which combines finite element and discrete element method is called FEM/DEM [12, 13, 14]. The method uses advantages of both approaches. Namely, the fragmentation process (strong discontinuity between elements) is modeled employing DEM, and the deformation in the element inside is modeled using FEM.

3. Analysis

The structure does not meet the criteria for regularity in the floor plan, as it has unevenly distributed openings and the mass is not symmetrically distributed relative to the horizontal axis. Additionally, not all gable walls are symmetrically positioned with respect to the vertical axis. The result of these asymmetries is eccentricity, which represents the distance between the center of mass and the center of rigidity (Fig. 2).

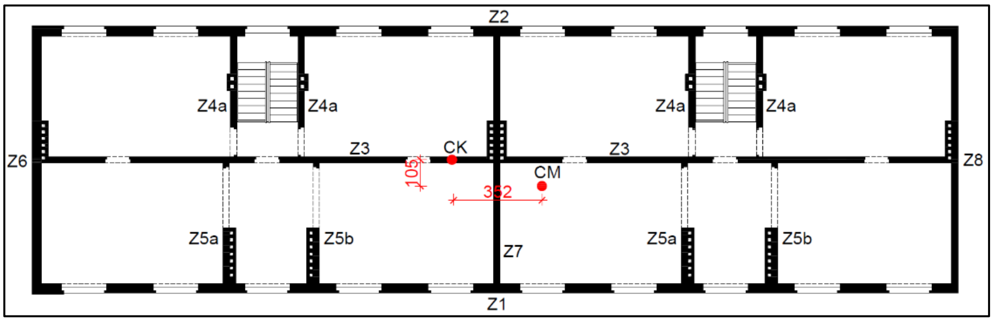

Figure 2. Center of mass (CM) and center of rigidity (CK).

Based on the seismic hazard map of the Republic of Croatia, the value of the reference peak ground acceleration (PGA) for a return period of 475 years was determined. For the residential building located in the Folnegovićevo neighborhood, the reference value is a_{gR} = 0.243g. The identification of the soil for the residential building cannot be determined with certainty, but soil category C has been selected, with a description and parameter values typical for this area.

The behavior factor *q* is an approximation of the ratio between the seismic forces that would act on the structure if its response were fully elastic with 5% relative viscous damping and the forces that can be used in the analysis with a conventional linear elastic model while still ensuring a satisfactory structural response. The ductility of the structure, which enables the dissipation of energy introduced by the earthquake, is accounted for using the behavior factor q = 1.5.

The methods for analysis of structures exposed to seismic action include:

- Linear methods (calculation using the equivalent static method and the response spectrum method)
- Nonlinear static analysis using the pushover method
- Linear and nonlinear dynamic analysis using earthquake accelerograms

Although linear analysis methods are permitted for the assessment of existing masonry buildings, preference is given to nonlinear analysis. Such analysis can be conducted using one of the previously mentioned approaches to structural modeling. The pushover method can also be applied to a detailed model. The choice depends on the capabilities of the numerical model and the assumptions made during its creation. Additionally, simple methods that provide a qualitative understanding of structural behavior are desirable, along with verification of the prescribed methods. One such method is checking the total surface area of load-bearing walls. However, this cannot be used to assess the building's overall load-bearing capacity. Verification is carried out based on limit state criteria for each component as well as

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at the global level. This means that a specific limit state is not satisfied if even one component fails to meet the criteria. Furthermore, an engineering assessment is required to ensure that a less significant element does not limit the load-bearing capacity of the entire building [15].

Linear elastic analysis for existing masonry buildings should be used with great caution. This is not only due to the difficulty in reliably estimating the behavior factor q but also because of the material properties and the distribution of sectional forces within the structure. Masonry elements have very low tensile strength, and the limit stress exceedance occurs even under small loads. Additionally, stress concentrations and singularities arise that are not realistic, meaning the stress values obtained through linear analysis are highly dependent on the density of the finite element mesh. Since linear analysis does not account for stress and force redistribution, an unrealistic force flow within the structure may be obtained, along with inaccurate sectional forces. For these reasons, linear elastic analysis is used primarily for preliminary assessments and should generally be avoided for detailed analysis of existing masonry structures.

When nonlinear methods are used for structural analysis, the force distribution varies in comparison to linear analysis [16]. This difference arises from the varying load-bearing capacities of walls, influenced by axial forces and bending moments. In nonlinear analysis, forces are distributed based on capacity as a function of displacement, while in linear analysis, force distribution is based on stiffness, independent of displacement magnitude.

In verifying the structure using linear elastic methods, if one load-bearing element exceeds its capacity, the limit state is not satisfied. An engineering assessment of the element's impact on the overall structural behavior and the building's global performance is necessary. Additionally, attention should be paid to the robustness of the structural system, ensuring that any potential load redistribution can relieve the overstressed element.

3.1.Equivalent static method

The equivalent static method (ESM) is the oldest and simplest procedure for calculating the seismic response of structures suitable also for “hand calculations”. It assumes that the building predominantly responds in its fundamental mode, and the effect of ground motion is represented by horizontal static forces applied at the floor levels, i.e., at points where the largest masses are concentrated. The entire structure can be represented as an equivalent cantilever beam. It can be conditionally stated that the cross-section of this equivalent cantilever beam corresponds to the building's floor plan. For buildings up to 40 m the fundamental period T_1 can be approximated by Eq. 1 which yields T_1 = 0.41 s (plateau of the response spectrum):

$$T_1 = C_t \cdot H^{3/4} \tag{1}$$

After distributing seismic forces on walls, the largest shear force acts on Z3d in X direction (V_x = 913 kN) and Z8 in Y direction (V_y = 2738 kN). Both walls fail to comply with load-bearing requirements for shear and bending implying damage in case of stronger ground motions.

The same ESM was applied in commercial software Tower 8 [17] assuming that the build has or has not tie beams (tie columns) (Fig. 3). Although regulations currently do not prohibit the use of this method, it cannot fully provide a high-quality analysis for seismic retrofitting. Masonry is a highly nonlinear material, and its stiffness under cyclic loading is not easily defined. In linear analysis, there is no redistribution of stresses and forces, resulting in an unrealistic flow of forces in the structure and, consequently, inaccurate internal forces. For these reasons, linear analysis should be used with caution. Modes of vibration are shown in Fig. 4. The first eigen-mode is translation in shorter transverse direction (Y) with f_1 = 3.1 Hz (f_{1c} = 3.5 Hz for the confined model), the second mode is translation in longitudinal direction (X) with f_2 = 3.8 Hz (f_{2c} = 3.9 Hz) and the third mode pertains to torsion with f_3 = 4.2 Hz (f_{3c} = 4.6 Hz) (Fig. 4).

The relevant sectional forces in the analyzed walls for the unconfined model amount to V_x = 889 kN (Z3d in X) and V_y = 2469 kN (Z8 in Y) which is almost equal to the “hand calculation”. The confined

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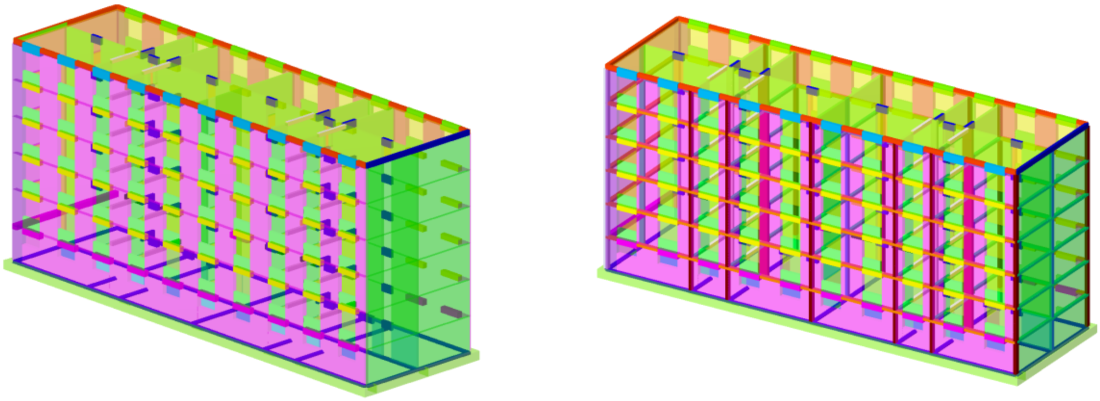
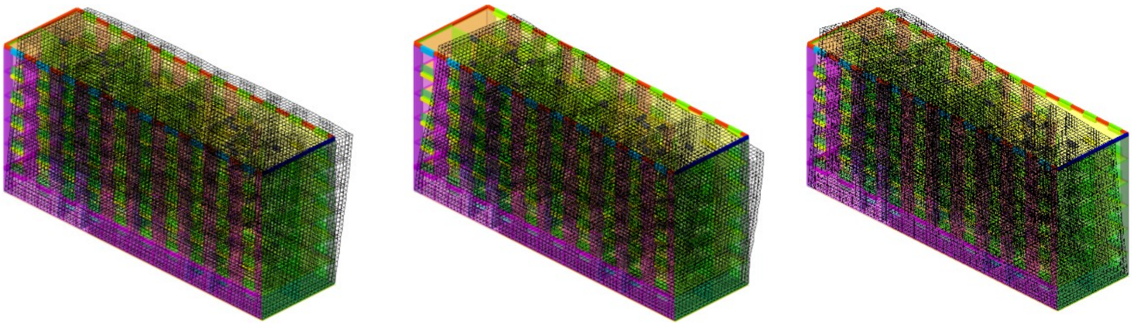
model yields reduced values V_x = 514 kN and V_y = 1482 kN which is attributed to the larger behaviour factor q = 2.5.



a) b)

Figure 3. 3D model of the building: a) w/o tie beams (columns), b) with confinement.



a) b) c)

Figure 4. Eigen-modes of the building: a) translation in Y, b) translation in X, c) torsion.

3.2. Linear time history analysis

The linear time history analysis was performed using the earthquake record from the Zagreb event on March 20, 2020 (Fig. 5). Accelerations were applied simultaneously in the X and Y directions, with a maximum acceleration of 2.196 m/s² in the N-S direction and 1.769 m/s² in the E-W direction, over a duration of 8 seconds with an integration time step of 0.01 seconds.

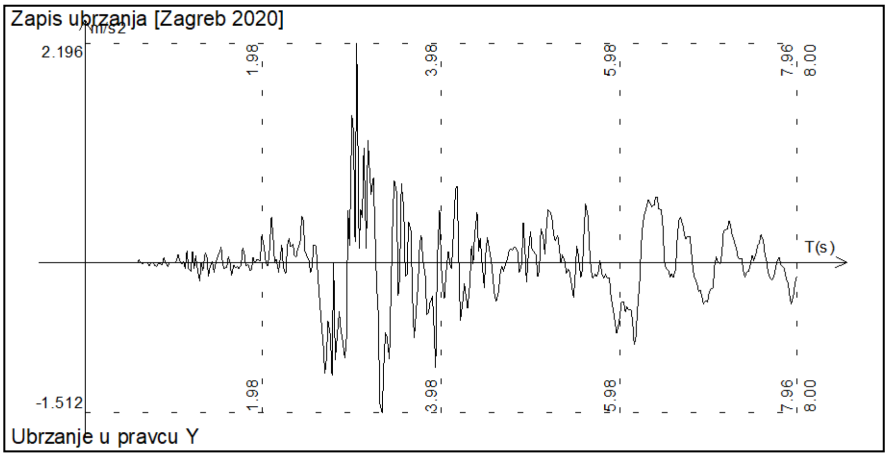


Figure 5. Acceleration record (N-S component) of Zagreb event.

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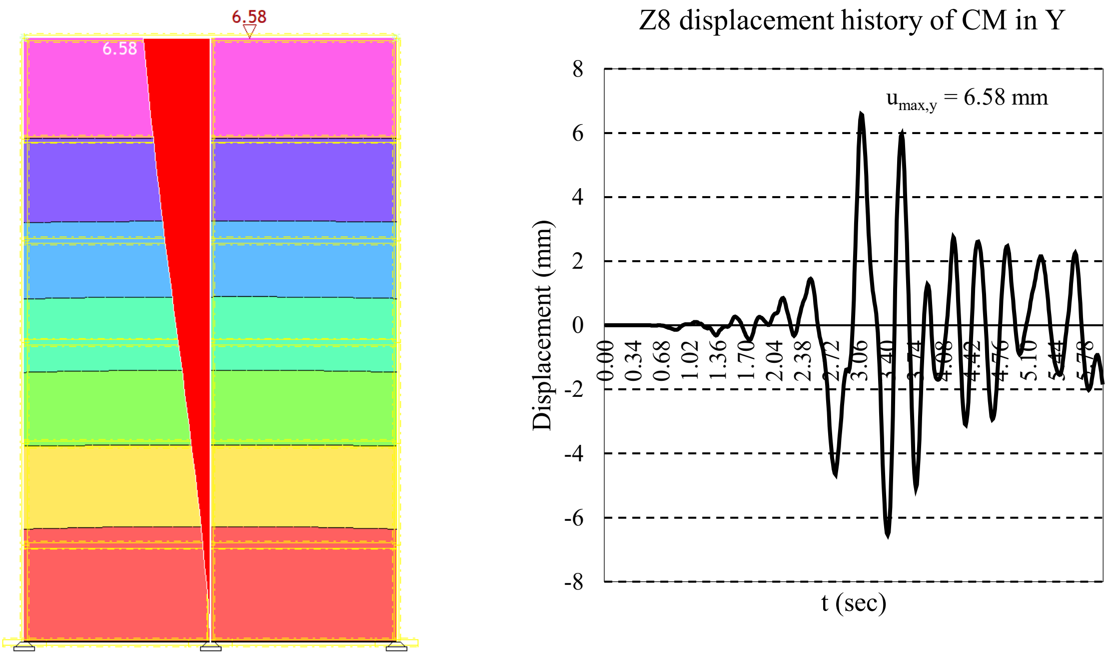
The displacement history of the center of mass (CM) at the top of the building is shown in Fig. 6. For the building without tie-beams/columns, the maximum displacement of CM in Y direction is $u_{max,y}$ = 6.58 mm (approximately H/2500, where the building height above the basement is H = 16.7 m) at $\ddot{u}_g$ = 2.196 m/s^2. For the confined building, the corresponding displacement is $u_{max,y}$ = 5.4 mm. The maximum shear forces for the unconfined building are V_x = 342 kN and V_y = 600 kN which are three to four times smaller than those obtained from ESM.



a) b)

Figure 6. Displacements in Y direction: a) maximum displacement of the CM for $\ddot{u}_g$ = 2.196 m/s^2, b) history of displacements of CM at the top of the building.

3.3. Nonlinear time history analysis

Nonlinear analysis was performed using DIANA. Engineering masonry constitutive model was employed was description of walls and the material parameters are listed in Tabel 1. Results of modal analysis are in excellent agreement with the previously shown model. Displacement history of the center of mass at the top of the building is shown in Fig. 7. The maximum displacement for the unconfined model equals to 13.2 mm (11.5 mm for the confined model) which is almost double the displacement obtained from the linear elastic THA. Interstory drifts are provided in Fig. 8 where it can be noticed that structural performance is far better than required by EC 8 or FEMA 356 for the Life-Safety Level. Number of cracks in integration point and crack widths are shown in Fig. 9. Crack widths of 3 mm and higher occur only locally, while the rest of the walls generally sustains the crack widths of 0.5 mm.

Table 1 – Parameters of the engineering masonry model for the analyzed building

Parameter	Value	Parameter	Value	Parameter	Value
E_x	3000 MPa	G_{ft}	10 N/m	G_{fs}	20 N/m
E_y	3000 MPa	HEADTP	NO	f_c	4,9 MPa
G_{xy}	1200 MPa	h	Rots	n	4
ρ	1600 kg/m^3	c	0,2 MPa	G_c	17000 N/m
f_{tx}	0,15 MPa	ϕ	32 °	λ	1

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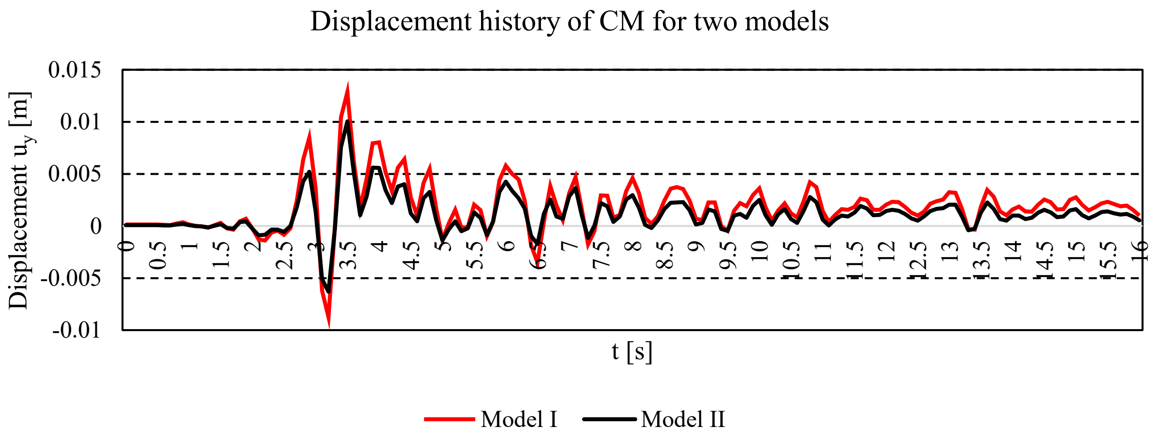


Figure 7. History of displacements of CM at the top of the building.

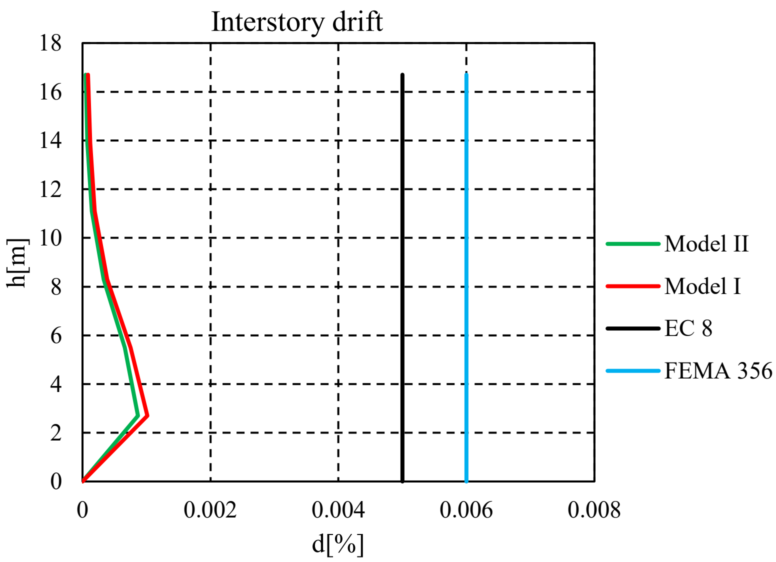
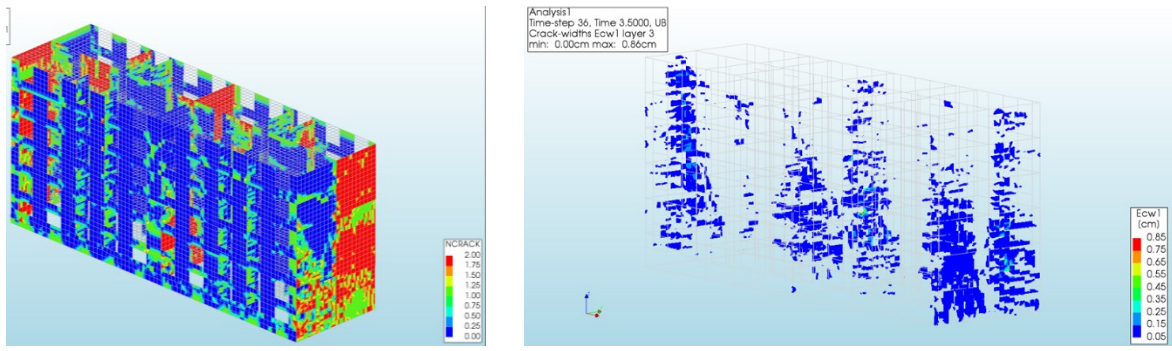


Figure 8. Interstory drifts for the analyzed building.



a) b)

Figure 9. Cracking parameters: a) NCRACK – number of cracks in integration point, b) crack width

3.4. Pushover analysis

The pushover analysis is a nonlinear static procedure where the lateral load on a structure increases monotonically until failure, while the load distribution remains constant. The load is applied in a

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predefined pattern, typically following the fundamental mode from elastic analysis. The relationship between the control node displacement (usually at the building's roof center of mass) and the base shear is plotted as a capacity curve (pushover curve). Fig. 10 presents pushover curves for two different models, showing base shear versus center-of-mass displacement. As expected, Model I, without confinement, exhibits lower stiffness and load-bearing capacity. Pushover analysis (PO) is a static method, whereas nonlinear time history (NLTH) analysis is dynamic and better represents seismic loading. Therefore, NLTH should be considered the reference, with the structural state from PO evaluated at the maximum displacement obtained from NLTH. While their overall behavior is expected to be similar, comparing PO and NLTH should focus on global response rather than direct results. This approach helps identify critical failure locations, determine failure causes, and provide insight into the structural response. Crack strains pertaining to opening (left) and crushing (right) of the gable wall Z8 are shown in Fig. 11.

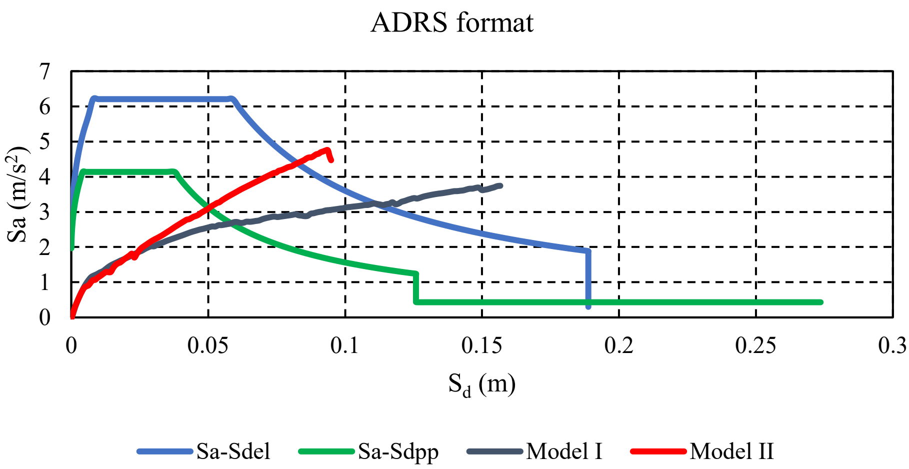

Figure 10. Pushover curves in ADRS format.

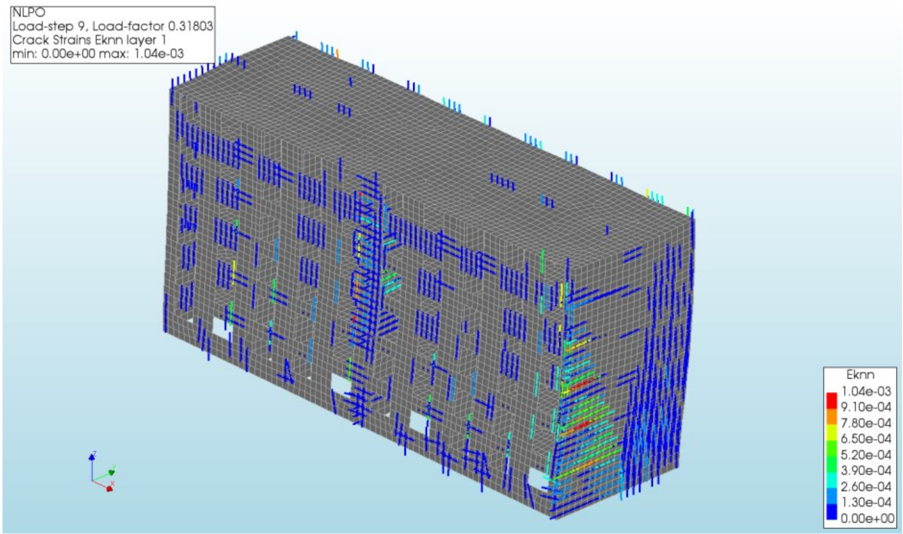

Figure 11. Crack strains in pushover analysis.

4. Damage categorization

To assess structural damage, engineers determine the damage severity (DS) level and evaluate its extent separately for each load-bearing direction. Based on this, the building's overall usability is assessed. Engineers rely on professional judgment, supported by guidelines, criteria, and reference photographs. The initial post-earthquake inspection template was based on Italian experiences. After the Zagreb

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earthquake, engineers struggled to classify buildings with damage levels II and III, similar to challenges in Italy. To improve assessments, additional criteria from Greek methodologies were introduced, providing clearer guidance on crack characteristics and their impact on usability, especially for masonry and reinforced concrete buildings. When categorizing damage, it is crucial to assess whether the building, despite reduced resistance, can withstand another earthquake of similar intensity.

The most widely referenced classification for damage assessment and building usability evaluations is based on the EMS-98 (European Macroseismic Scale), commonly used to determine earthquake intensity [18]. Masonry buildings exhibit significant structural diversity due to variations in materials, construction methods, and structural elements. Key factors to consider include wall condition, floor structure, arches, vaults, roof properties, and hazardous non-structural elements [15].

Considering crack types and maximum widths, Model I (8.5 mm in Y, 4.53 mm in X) aligns with damage level III, while Model II (6.7 mm in Y, 2.61 mm in X) corresponds to damage level II under EMS-98.

5. Conclusion

The conclusions of this study highlight key insights into the seismic performance of a typical pre-code unreinforced masonry (URM) building in Zagreb and offer recommendations for future assessments and retrofitting. Linear elastic analysis, while useful for preliminary assessments, tends to be overly conservative. It often predicts failure at force levels far below the actual capacity of the structure due to the inability to account for stress redistribution and plastic deformations. Because masonry has low tensile strength, this method frequently results in unrealistic stress concentrations, making it unreliable for detailed seismic evaluations.

Nonlinear analysis methods, such as Nonlinear Time-History Analysis (NLTH) and Pushover Analysis, provide a much more accurate representation of structural behavior under seismic loads. These approaches allow for stress redistribution, capturing the effects of material degradation, plasticity, and progressive damage, thereby offering a more realistic assessment of seismic performance. The study compared two structural configurations: an unconfined masonry model without tie beams and a confined model with tie beams. The unconfined model exhibited higher displacement and concentrated damage in specific areas, making it more vulnerable to shear and bending failures during strong ground motions. On the other hand, the confined model demonstrated better seismic performance by reducing displacement, improving force distribution, and increasing ductility, making it more compliant with modern seismic safety standards.

To classify the extent of damage, the study referenced the European Macroseismic Scale (EMS-98). The unconfined model aligned with Damage Level III, indicating moderate to substantial structural damage, while the confined model was categorized as Damage Level II, suggesting moderate damage but still being structurally usable with appropriate repairs. The study also incorporated damage classification methodologies from Greece and Italy to enhance accuracy in post-earthquake assessments.

Based on the findings, several retrofitting strategies are recommended to improve seismic resilience. The addition of horizontal and vertical tie beams can significantly enhance the building's ductility, reducing the risk of sudden failure. Strengthening connections between floor slabs and walls would improve overall force distribution, while reinforcing critical load-bearing walls would help mitigate failure risks. Moreover, conducting site-specific soil investigations is essential to refine seismic hazard assessments and improve future structural evaluations.

This research contributes to the methodology for assessing and strengthening URM buildings, providing a framework that can be applied to similar structures in seismic-prone regions. It underscores the importance of employing advanced nonlinear analysis techniques over traditional linear methods to achieve more reliable seismic evaluations. Additionally, the findings could support updates to policies

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and regulations governing the retrofitting of pre-code masonry buildings in Croatia and other regions facing similar challenges.

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