Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

NONLINEAR STATIC AND DYNAMIC ANALYSIS OF A TYPICAL MASONRY BUILDING IN PALMOTIĆEVA STREET IN ZAGREB

Vanesa Jusufbašić (1), Senad Medić (2), Mario Uroš(3)

(1) *Mag.ing.aedif., IPSA Institute, vanesa.jusufbasic10@gmail.com*
(2) *Assistant professor, University of Sarajevo – Faculty of Civil Engineering, senad_medic@yahoo.com*
(3) *Associate professor, University of Zagreb – Faculty of Civil Engineering, mario.uros@grad.unizg.hr*

Abstract

In this study, nonlinear static pushover and dynamic time-history analyses of a typical masonry building situated on Palmotićeva street in downtown Zagreb were performed. The building was erected in 1922 before any seismic codes were introduced in practice. It has a basement, four stories, an attic (total height equal to ca. 23m), and an asymmetric plan consisting of two connected parts: a street part (24.4x12 m) and a courtyard part (10.6x12 m). The floor structure consists mainly of wooden beams except above the basement, where the RC slab was installed. The solid brick masonry walls with variable thicknesses (15-90 cm) are evenly distributed in both directions. Two numerical macro-models were created employing Diana 10.4. Engineering masonry constitutive law was used to describe the highly nonlinear behavior of masonry walls which can crush, crack or fail in shear. Three numerical models were created describing the current damaged state and the possible strengthening with rigid floor diaphragms. The response of the building was assessed in terms of capacity curves, inter-story drifts, and cracking patterns.

Keywords: damaged building, engineering masonry model, nonlinear analysis, seismic capacity

1. Introduction

Unreinforced masonry (URM) structures are made of brick units connected by mortar joints. The behavior of a masonry structure strongly depends on the type of brick, mortar composition, brick dimensions, and the way bricks are assembled [1]. In general, brick and mortar perform well under compression, but their tensile capacity is considerably lower. Three different in-plane failure mechanisms can occur in URM walls: diagonal cracking, shear sliding, and rocking [2,3]. Diagonal cracking and sliding are failure modes caused by shear, and rocking is a consequence of flexural behavior. The occurrence of different failure modes depends on the geometry of the pier, boundary conditions, axial load, mechanical properties of masonry, and geometrical characteristics of masonry. Several experimental tests have been performed to find the relation between different failure modes and the mentioned parameters. Generally, it has been concluded that rocking tends to occur in slender piers with lower precompression, shear sliding in squat piers, and diagonal cracking in moderately slender piers [4, 5].

The Republic of Croatia is among the most earthquake-prone countries in Europe, yet the current activities related to assessing potential earthquake risk and its reduction can be characterized as individual and insufficient [6]. The city of Zagreb has a moderate seismic hazard, but it is highly exposed (densely populated), and the built environment is quite vulnerable, meaning the seismic risk is high. A severe earthquake hit Zagreb on March 22, 2020 (magnitude ML = 5.5, with an epicenter 7 km north of the city center). The event occurred during the COVID-19 lockdown and caused significant damage to the built environment and enormous disruption in everyday life [7].

Since the old URM typologies were not designed to withstand seismic loads, they were heavily affected by the earthquake. In addition to the design characteristics, their age and often inadequate maintenance contributed to the poor performance of these buildings. Aggravating factors are the subsequent renovations, upgrades, and changes in function. The original up to 60 cm thick solid brick load-bearing walls, composed of two to three rows of molded clay bricks, are often reduced in thickness or partially

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

or even entirely removed at the ground level to install street store windows. Steel lintels are frequently installed to span new or extended openings. In certain cases, the upper floors' partitions and interior bearing walls are entirely disregarded as part of the structure and removed to gain space. Such interventions result in unsupported walls, initially continuous in the vertical direction, or out-of-plane critical walls, significantly weakening the structural system. These interventions are seldom documented, and the current condition of the building differs significantly from the original documentation [8, 9, 10].

Accurate assessment of masonry buildings is a challenge due to the nonlinear behavior of masonry and the dynamic nature of a seismic load. A Nonlinear Time History (NLTH) analysis considers both factors, but the computer resources and huge amount of time limit its everyday use. Another widely used method is the Modal Response Spectrum method, where the nonlinear material response is considered indirectly via a behavior factor. The results of the Response Spectrum method are considered too conservative [11]. A third method is the Nonlinear Pushover (NLPO) method. It considers nonlinear material behavior, and compared to NLTH, NLPO is computationally more efficient. An equivalent lateral load pattern is applied in a quasi-static fashion producing a capacity curve that is subsequently compared to demand in terms of an acceleration-displacement response spectrum. Pushover analysis is a practical alternative because it gives good insight into the seismic response.

The structure's performance is studied by looking into the force-displacement response, displacement profile, and damage pattern. Another aspect that requires attention is the different modeling strategies used to execute an analysis [12,13]. This paper employed the continuum finite element approach by assuming shell elements in DIANA [14].

The goal of the case study is to numerically describe the response of a typical unreinforced masonry building located at 64a Palmotićeva Street in Downtown Zagreb. All results pertain to the N-S direction of the ground motion (X-axis of the building).

2. Building Description

The building was built in 1922 and has a basement, ground floor, three floors, and an attic (Fig. 1). The plan dimensions of the building are 24.40x12 m (street part) and 10.6x12 m (courtyard part), and the floor area is about 407 m^2. The total gross floor plan area of the building is 2440 m^2, while its total height is 22.70 m (6x3.5+2.9), i.e., the building extends from -1.2 to 22.70 m. The street and courtyard sections of the building are connected and form one unit, but this is also the cause of asymmetry. The building meets the criterion of regularity in elevation, while the criterion of regularity in the plan is not met. A heterogeneous horizontal load-bearing structure does not act as a rigid floor diaphragm. Hence, an unfavorable and irregular structural response, where parts of the building behave independently, is expected during an earthquake. The structure consists of connected solid brick walls extending continuously from the foundation to the roof.

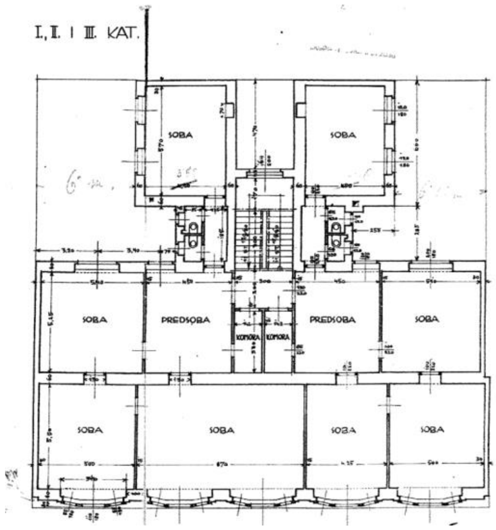
The walls are evenly spaced in both directions. Load-bearing walls are made of the old format brick (290x140x65mm) and have a variable thickness (90, 65, 45, or 15 cm). Partition walls are made of solid brick with 7- and 15-cm thicknesses. The walls are tied by lintels, parapets, and beams, the composition and quality of which are not fully known. The parapets and lintels on the facade openings are thinner than the connecting walls and are usually 30 cm. The horizontal structure above the street part of the basement is an RC slab with a system of RC beams. Above the ground floor and upper floors, the structure consists of wooden beams with planks and loose filling inside the deck structure. On the south side of the building above the ground floor and the third floor, during subsequent reconstructions, RC slabs with a thickness of 8 cm were constructed and coupled to wooden beams. The roof structure is wooden and gabled, and the attic was converted into a living space over time. The building has an internal U-shaped staircase made of prefabricated RC elements supported on the walls and steel profiles with RC landings.

2CroCEE
22 - 24 March 2023

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023
Copyright © 2023 CroCEE
DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

Faculty of Civil Engineering in Zagreb conducted the experimental investigation of mechanical properties of masonry and ambient vibration tests, the results of which were used as input parameters and calibration of numerical models [10].

a)



b)

Figure 1. a) View of the building; b) Layout of the typical story (Courtesy of Zagreb City Archives).

3. Numerical Modeling

3.1 Engineering masonry material model

Creating powerful micro-models, in which each constituent of composite masonry is described separately (brick, mortar, interface), is often not feasible in practice. Besides difficulties related to individual material properties, the considerable computational time is required even for small-scale models. The approach based on averaged constitutive equations seems suitable for large-scale finite element analyses by collating experimental data at an average level (macro-modeling) or from homogenization techniques [15]. A popular model for the simulation of quasi-brittle materials such as concrete and masonry where cracking is smeared over the finite element is the Total Strain Crack model [16]. However, two shortcomings when masonry is cyclically loaded were noted in the literature [17]. The model was derived for isotropic materials, and the secant reloading curves underestimate the energy dissipation under cyclic conditions.

A type of masonry model that avoids the limitations of the Total Strain Crack model is the Engineering Masonry model (ENGMAS) [17]. This 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 [18]. The Engineering Masonry model describes the unloading behavior more realistically than the Total Strain Crack model. It assumes a substantial stress decay with the initial linear stiffness. Anisotropy is included by considering different stiffness in the direction of the bed and head joints. Stresses in both directions are defined by their respective strain components and the maximum value of the strain that has been reached in the lifetime of an element.

3.2 Model properties

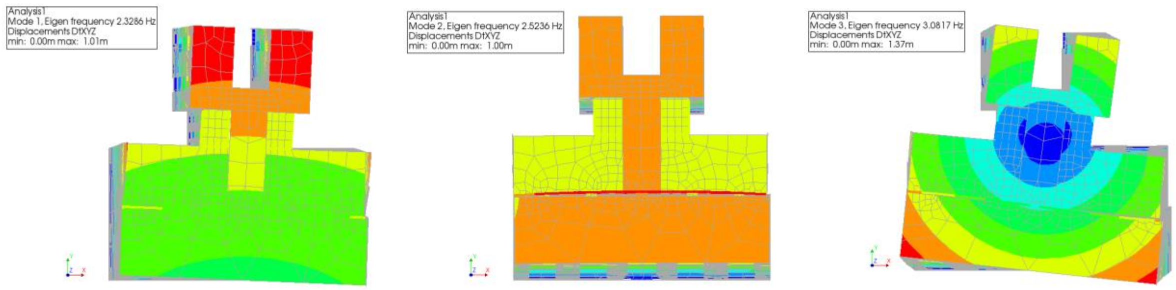
Three models with respect to the horizontal load-bearing structure were created: model 1 – rigid floor diaphragms, model 2 – wooden beams, and model 3 – modified RC floors coupled with wooden beams above the ground floor and the third floor (reconstruction). The gravity load is assumed to be 2.5 kN/m2, and the live load equals 1.5 kN/m^2. Walls and slabs are discretized with curved shell finite elements CQ40S and CT30S with a high integration scheme [14]. The floor structure is elastic, while masonry is described with the ENGMAS material model. The properties of masonry are listed in Table 1.

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

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

Parameter	Value	Parameter	Value	Parameter	Value
E_x	1500 MPa	G_{ft}	10 N/m	G_{fs}	20 N/m
E_y	1500 MPa	HEADTP	NO	f_c	3,4 MPa
G_{xy}	500 MPa	h	Rots	n	4
ρ	1800 kg/m^3	c	0,16 MPa	G_c	16000 N/m
f_{tx}	0,114 MPa	ϕ	32 °	λ	1

The presented modeling strategy was validated by comparing analytical, numerical, and experimental results obtained from cyclic static tests of URM cantilever walls under constant vertical precompression [19, 20]. Vibration modes for model I are shown in Fig. 2. Eigenfrequencies for the three models in the initial (no damage) state are listed in Table 2. The reduction of frequencies for model II with wooden floor beams is as expected, while the eigenmodes basically retain the same shape for all models.



a) b) c)

Figure 2. Eigen modes for model I: a) mode I, f_1 = 2,32 Hz; b) mode II, f_2 = 2,52 Hz; c) mode III, f_3 = 3,08 Hz.

Table 2 – Eigen frequencies of the three models

Model	f_1 [Hz]	f_2 [Hz]	f_3 [Hz]
I	2,32	2,52	3,08
II	2,08	2,39	2,71
III	2,13	2,43	2,80

3.3 Nonlinear time history analysis

In NLTH, the seismic load is considered by applying a ground motion signal to the soil or directly to a structure. Several earthquake signals should be applied to scrutinize the substantial spread an earthquake scatter could have at a single location. Both material and geometric nonlinearity are considered. The analysis was executed for 38-sec duration of the Zagreb earthquake by applying the signal recorded at the Office of emergency management of the City of Zagreb. The PGA of the N-S component was 0.22 g, whereas the peak ground acceleration of the E-W component amounted to 0.179g (Fig. 3). The model is shown in Fig. 4a, and the crack status for model II is provided in Fig. 4b.

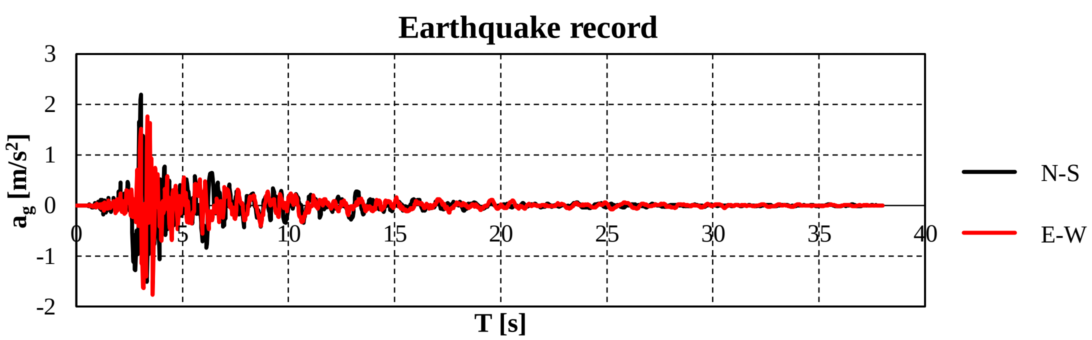
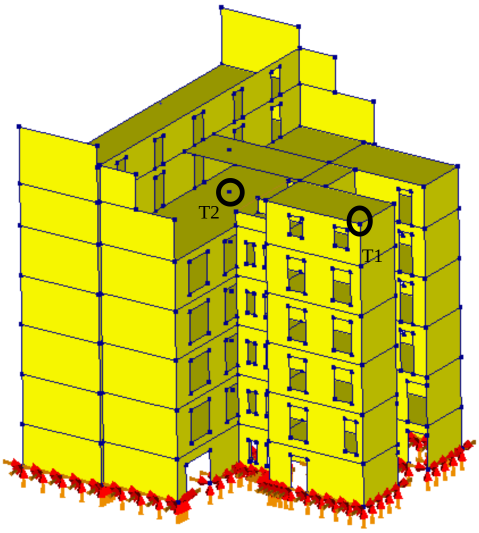


Figure 3. Zagreb earthquake record (March 22, 2020).

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023
Copyright © 2023 CroCEE
DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99



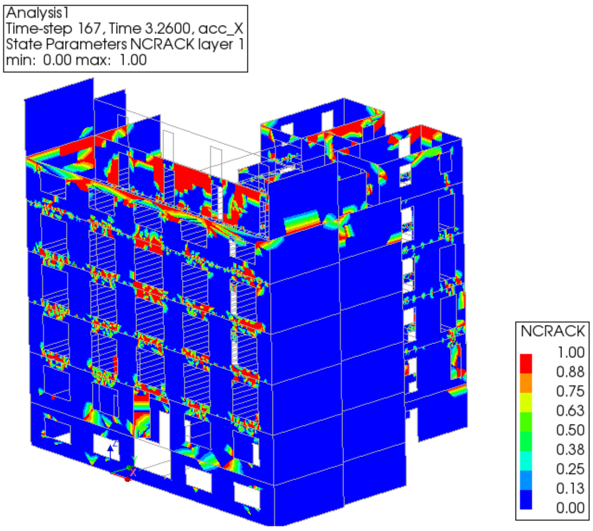


Figure 4. a) Model of the building with control points; b) Crack status for model II.

Crack distribution should serve as an indication of potential damage or weak spots (e.g., lintels in Fig. 4b). The displacement history of the center of mass (point T2 of model II) for the first 7 seconds of earthquake duration is given in Fig. 5. Maximum displacements of the control point T2 at h = 20m for all three models are listed in Table 3. The maximum attained displacement is approximately 4 cm. Maximum relative displacements (d/h), e.g., inter-story drifts, are shown in Fig. 6.

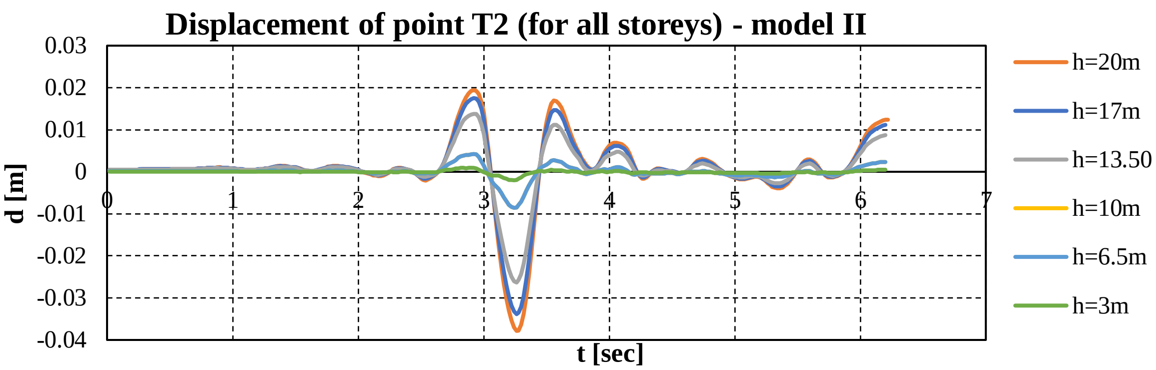


Figure 5. Displacement history of point T2 (center of mass) for model II (d_{max} = 3.8 cm).

Table 3 – Maximum displacements of the three models

Model	d [cm] – T1	d [cm] – T2
I	3,88	3,12
II	3,88	3,78
III	4,09	3,94

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023
Copyright © 2023 CroCEE
DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

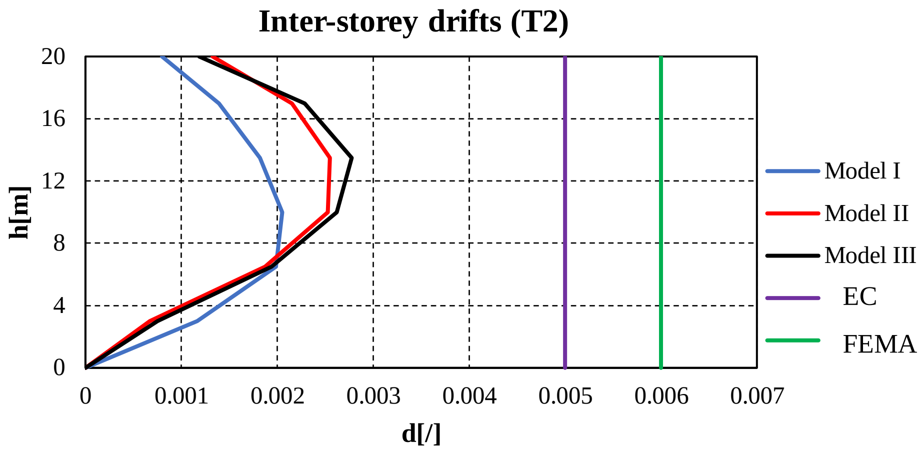
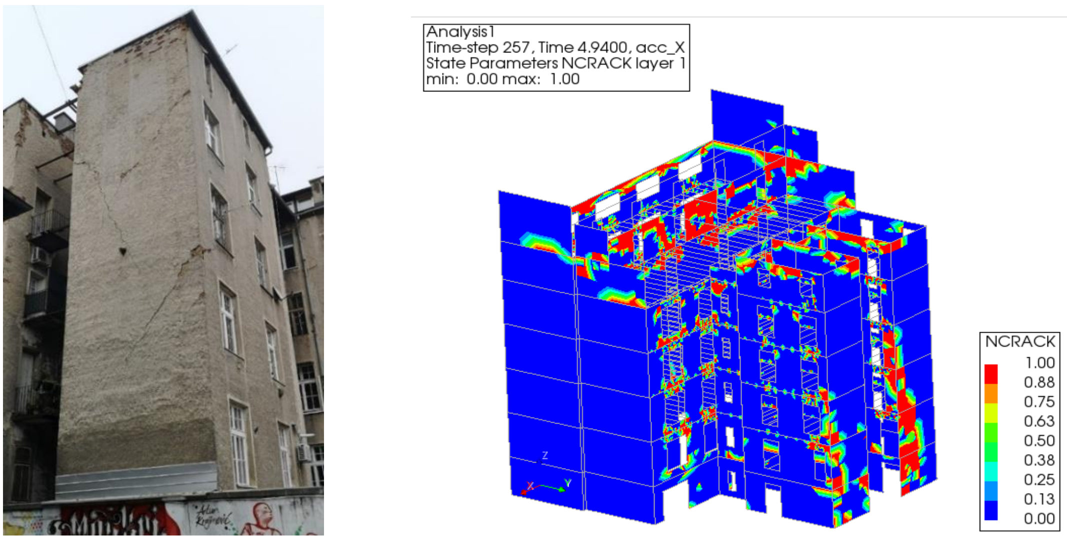


Figure 6. Maximum inter-story drifts.

Fig. 7 shows the western courtyard wall that was most damaged in the earthquake. It also compares the actual state and the results of numerical modeling. It has long diagonal cracks that run along the entire wall and end at the side window openings. The width of the cracks is up to 15mm - which means that the wall has failed and represents a great danger for the building and the tenants in case of subsequent earthquakes. The causes of the degradation of the walls are related to the following:

a) The floor structure is not anchored to the perimeter walls; the beams are parallel to the western walls. There is a thin concrete slab, but the connection with the perimeter and western walls is missing. Additionally, the western wall is connected to perpendicular walls, which are weakened by large openings.
b) The western wall is exposed to the highest seismic demand being far from the stiffness center.



a) b)

Figure 7. Degradation of the building: a) Actual state after Zagreb earthquake; b) Cracking pattern.

3.4 Pushover method

The pushover analysis is a nonlinear static procedure in which the magnitude of the lateral load excited in a structure increases monotonically until failure, while the load distribution remains constant. The lateral load is applied in a predefined load pattern that follows the fundamental mode from the elastic analysis. The relation between the control node displacement (usually the center of mass of the roof of the building) and base shear is plotted subsequently in a so-called capacity curve (pushover curve).

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

Pushover curves for three different models are provided in Fig. 8 (base shear vs. displacement of the center of mass CM/T2). As expected, the model I with rigid diaphragms representative of possible strengthening has the largest stiffness and load-bearing capacity.

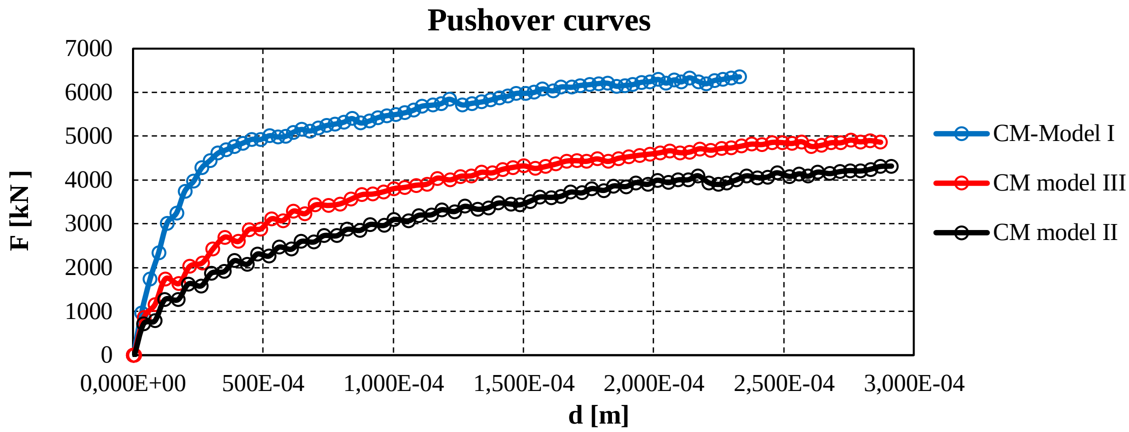


Figure 8. Pushover curves for three different models.

This capacity curve is used to determine the seismic capacity of a structure. The earthquake demand is represented by the smoothed elastic response spectrum, which is formed considering the peak acceleration of the soil and the soil category. The design spectrum can be formed by introducing the behavior factor q=1.5 (most common for masonry structures), and thus reduce the earthquake requirement by considering the expected plastic deformations. The pushover curve and the response spectrum need to be transformed into the capacity spectrum using the structure's originally elastic dynamic properties (participation factor and modal mass) to compare the capacity with demand. This capacity spectrum is represented in the Acceleration Displacement Response Spectrum format (ADRS), using spectral displacements ($S_d = S_a/\omega^2$) and spectral accelerations (S_a) (Fig. 9). The intersection of the capacity curve with the spectral curve is called the performance point.

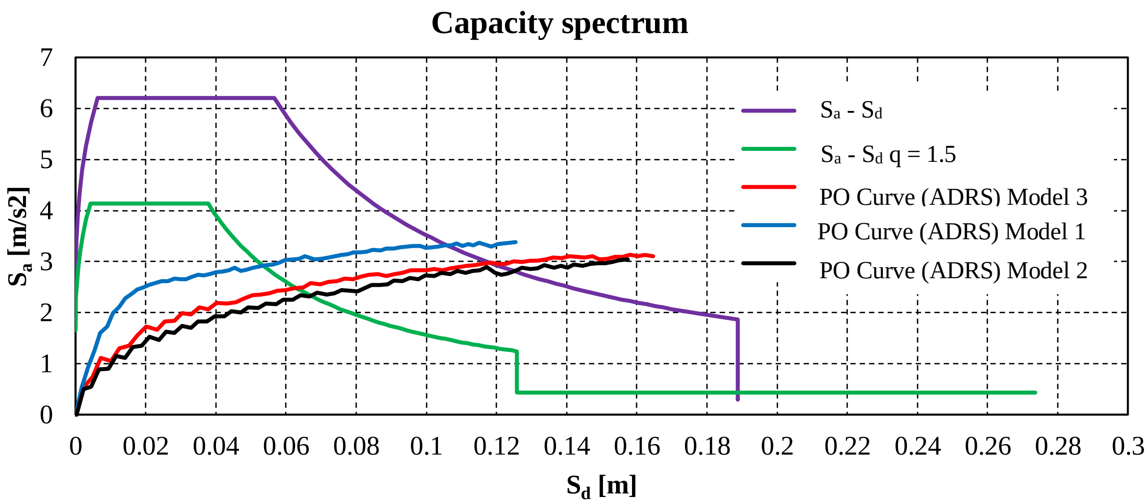


Figure 9. ADRS format.

3.5 Influence of element type

The influence of the finite element type is investigated by choosing linear and quadratic interpolation functions while keeping the average element area the same. In addition to previously shown results related to quadratic polynomials, triangular T15SH and rectangular Q20SH finite elements with linear shape functions are employed for pushover and NLTH analysis. The results show that linear elements

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

are generally stiffer and yield a larger load-bearing capacity. The difference in pushover curves for the two models is provided in Fig. 10. The same is valid regarding the NLTH and inter-story drifts. On the other hand, due to the reduced number of degrees of freedom for linear elements, the computational time is shorter and convergence faster.

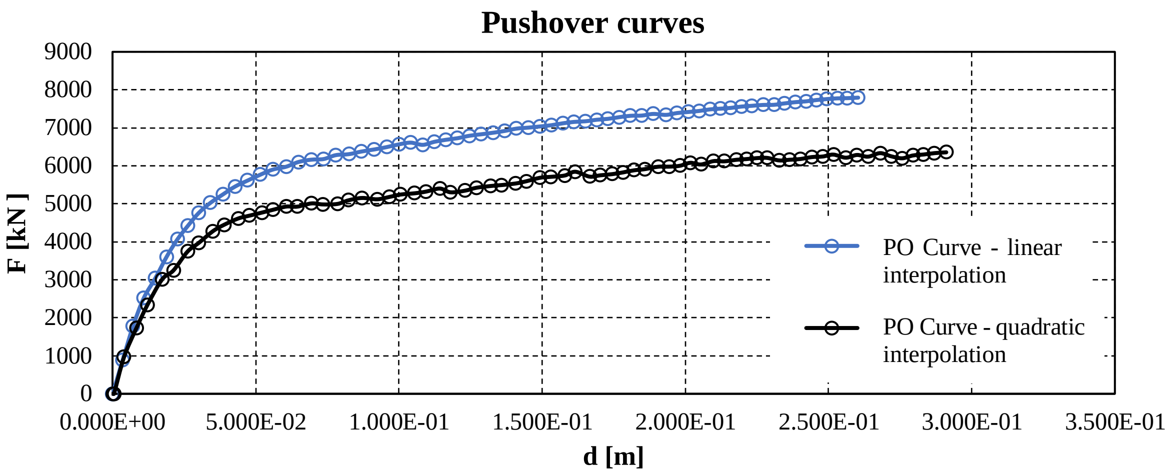


Figure 10. Pushover curves for linear and quadratic interpolation functions.

4. Conclusion

This paper employed DIANA FEA to conduct a nonlinear static and dynamic analysis of the existing residential masonry building in Zagreb Lower Town. The geometry of the building was adopted after drawings were made available by the Zagreb City Archives. Material properties and essential dynamic characteristics were taken from experimental investigations conducted by the Faculty of Civil Engineering in Zagreb. Recently developed Engineering Masonry Model was used to describe walls in a continuum macro model. Three models with respect to the horizontal load-bearing structure were created: model 1 – rigid floor diaphragms, model 2 – wooden beams, and model 3 – modified RC floors coupled with wooden beams above the ground floor and the third floor (reconstruction).

Nonlinear dynamic analysis implies the application of ground acceleration records at the base level. This method requires more input parameters, memory, and computer speed. Therefore, it is limited for everyday engineering applications. On the other hand, true dynamic behavior can be examined at any time instance. The calculations for a limited earthquake duration (1/5 of the total record) lasted three days. Pushover analysis was proven helpful in monitoring the formation and distribution of cracks in load increments and controlling the global structural response. Limitation to buildings of a regular shape without particular eccentricities is the disadvantage of this method. The dynamic calculation should be taken with a grain of salt due to the incomplete input data required for such a calculation, but also the possibility that the ground motion records used did not sufficiently shake the building, leaving the possibility that another earthquake with the same peak ground acceleration could have a more devastating effect. As expected, the analysis showed that the model with RC slabs is the stiffest compared to the other two.

The cracks occur in areas where the initial shear strength is reduced and the low tensile strength is exceeded (corners around the openings, joints of the walls, lintels, and junction of the main and the central/courtyard section). Damage is evident, especially on the courtyard wall. In the model with beams, cracks are visible at the contact of floors and walls. A rigid or partially rigid diaphragm is essential for favorable earthquake response. Damage is mainly concentrated on the higher floors due to the smaller thickness of the walls and lower compressive stress.

Determining the stiffness of walls and lintels is a problem even for vertical loads, especially for cyclic seismic loads. The change in stiffness occurs in larger areas or locally due to cracking, crushing, or

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

sliding. Modal analysis was executed after NLTH, and the obtained fundamental frequency matches the experimental determined by ambient vibration tests after the earthquake.

The influence of finite element type was investigated by choosing linear and quadratic interpolation functions while keeping the average element area the same. The results show that linear elements are generally stiffer and yield a larger load-bearing capacity. On the other hand, due to the reduced number of degrees of freedom for linear elements, the computational time is shorter and convergence faster.

Acknowledgements

The authors wish to acknowledge the support of this research effort by the Croatian Science Foundation (grant number UIP-2020-02-1128).

References

[1] Mosalam, K., Glascoe, L., Bernier, J. (2009): *Mechanical Properties of Unreinforced Brick Masonry, Section1* (No. LLNL-TR-417646). Lawrence Livermore National Lab. (LLNL), Livermore, CA (United States).

[2] Petrovčič, S., Kilar, V. (2013): Seismic failure mode interaction for the equivalent frame modeling of unreinforced masonry structures. *Engineering structures*, *54,* 9-22, doi: http://dx.doi.org/10.1016/j.engstruct.2013.03.050

[3] Mistler, M. (2006): *Verformungsbasiertes seismisches Bemessungskonzept für Mauerwerksbauten* . Mainz.

[4] Magenes, G., Calvi, G. M. (1992): Cyclic behaviour of brick masonry walls. In *Proceedings of the 10th world conference on earthquake engineering* (pp. 3517-3522).

[5] Tomazevic, M. (1999): *Earthquake-resistant design of masonry buildings* (Vol. 1). World Scientific.

[6] Atalić, J., Šavor Novak, M., Uroš, M. (2019): Seismic risk for Croatia: Overview of research activities and present assessments with guidelines for the future. *Građevinar*, 71(10.), 923-947, doi: https://doi.org/10.14256/JCE.2732.2019

[7] Stepinac, M., Lourenço, P. B., Atalić, J., Kišiček, T., Uroš, M., Baniček, M., Novak, M. Š. (2021): Damage classification of residential buildings in historical downtown after the ML5. 5 earthquake in Zagreb, Croatia in 2020. *International journal of disaster risk reduction,* 56, 102140, doi: https://doi.org/10.1016/j.ijdrr.2021.102140

[8] Atalić, J., Uroš, M., Šavor Novak, M., Demšić, M., Nastev, M. (2021): The Mw5. 4 Zagreb (Croatia) earthquake of March 22, 2020: impacts and response. *Bulletin of Earthquake Engineering*, 19(9), 3461-3489, doi: https://doi.org/10.1007/s10518-021-01117-w

[9] Novak, M. S., Uros, M., Atalic, J., Herak, M., Demsic, M., Banicek, M., Lazarevic, D., Bijelic, N., Crnogorac, M., Todoric, M. (2020): Zagreb earthquake of March 22 2020-preliminary report on seismologic aspects and damage to buildings. *Gradevinar*, 72, 843-867, doi: https://doi.org/10.14256/JCE.2966.2020

[10] Uroš, M., Todorić, M., Crnogorac, M., Atalić, J., Šavor Novak, M., Lakušić, S. (2021): *Potresno inženjerstvo-Obnova zidanih zgrada.* Građevinski Fakultet, Sveučilišta u Zagrebu: Zagreb, Croatia.

[11] Wilson, E.L. (2013): *Educational Priorities of an Old Professor on Seismic Analysis of Structures.* http://www.edwilson.org/History/Educational%20Priorities%20of%20an%20Old%20Professor.pdf

[12] D'Altri, A. M., Sarhosis, V., Milani, G., Rots, J., Cattari, S., Lagomarsino, S., Sacco, E., Tralli, A., Castellazzi, G., de Miranda, S. (2020). Modeling strategies for the computational analysis of unreinforced masonry structures: review and classification. *Archives of computational methods in engineering*, *27*, 1153-1185, doi: https://doi.org/10.1007/s11831-019-09351-x

[13] Medić, S., Hrasnica, M. (2018): Modeling strategies for masonry structures. *In Advanced Technologies, Systems, and Applications II: Proceedings of the International Symposium on Innovative and Interdisciplinary Applications of Advanced Technologies (IAT)* (pp. 633-644). Springer International Publishing, doi: https://doi.org/10.1007/978-3-319-71321-2_55

[14] DIANA FEA (2016): *DIANA - User's Manual, Material library*, Delft, The Netherlands.

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.99

[15] Lourenço, P. B. (2015). Masonry modeling. *Encyclopedia of Earthquake Engineering*, 1-13, doi: https://doi.org/10.1007/978-3-642-36197-5_153-1

[16] Rots, J. G., Blaauwendraad, J. (1989): Crack models for concrete, discrete or smeared? Fixed, multi-directional or rotating?. *HERON, 34 (1), 1989*.

[17] Schreppers, G.J., Garofano, A., Messali, F., Rots, J.G. (2016): *DIANA Validation report for Masonry modelling*, DIANA FEA BV and TU Delft, Delft, The Netherlands.

[18] Noortman, F. (2019): *Applicability of the Pushover Method for the Seismic Assessment of URM Structures in Groningen: A Case Study of a Low-Rise Apartment Building*, TU Delft, Delft, The Netherlands.

[19] Jusufbašić, V. (2021): *Nelinearna statička i dinamička analiza stambene zgrade u Palmotićevoj ulici*, Master thesis (in Bosnian). University of Sarajevo – Faculty of Civil Engineering, Sarajevo, Bosnia Herzegovina.

[20] Medić, S., Hrasnica, M. (2021): In-Plane Seismic Response of Unreinforced and Jacketed Masonry Walls. *Buildings*, 11(10), 472, doi: https://doi.org/10.3390/buildings11100472