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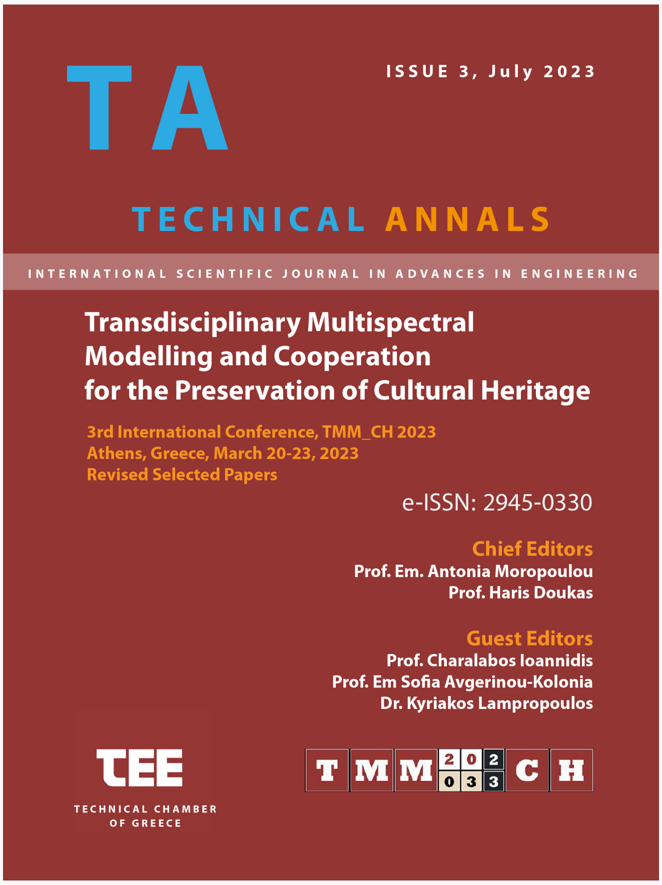

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Post-earthquake assessment of monumental building in Croatia by 3Muri software

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Abstract. A series of earthquakes in and close to city of Zagreb, Croatia during the 2020 severely damaged the masonry cultural heritage buildings built in the 19th century. The article presents the assessment of post-earthquake design of retrofitting of damaged historic masonry structure of the former synagogue in city of Sisak, Croatia erected in 1890. The former synagogue was severely damaged during the recent earthquake. For the purposes of the post-earthquake retrofitting and strengthening, an analysis of the damaged and retrofitted structure has been carried out to justify the selected retrofitting measures that included a partial reconstruction of heavily damaged structure. The equivalent frame model based on the discretization in terms of piers and spandrels was created (SEM - Structural Elements Model) to obtain the simulation of earthquake response of structure using the 3Muri 13.9 software. In this paper the procedure and results of the analysis of seismic resistance of the strengthened structure is presented because of the successful cooperation of structural engineers, architects and conservators.

Keywords: Earthquake 2020 Croatia, Monumental Building, Retrofitting, Structural Elements Model, Assessment, 3Muri program

1 Introduction

The year of 2020 was the year of serious earthquakes affecting the north-western part of Croatia including capital Zagreb. On 22. March 2020 the M_W 5.4 earthquake with epicenter in the northern suburban scattered populated hilly area of Zagreb was the first in row of earthquakes that culminated with 55 km south-east M_L 6.2 earthquake close to the village of Strašnik in Banovina region of Croatia. According to [1] the accelerograms of Strašnik earthquake has been recorded in 6 seismograph stations located in Zagreb area in average distance of 52,5 km. The highest PGA amplitudes were derived from the records of station named QKAS located in Zagreb area 57.8 km north of epicenter. The value of the horizontal south-north direction PGA (pick ground acceleration) was 0.248g and the value of the vertical PGA was 0.125g. The values of corresponding PGAs obtained from accelerograms recorded in other 5 stations were roughly 50% of the QKAS recorded values. Author [1] explains the reason for differences with the geological characteristics of locations where stations are located. In case of QKAS they may be identified as ground type C according to Eurocode 8, while on other 5 locations the ground properties correspond type A (see Fig.1(a)).

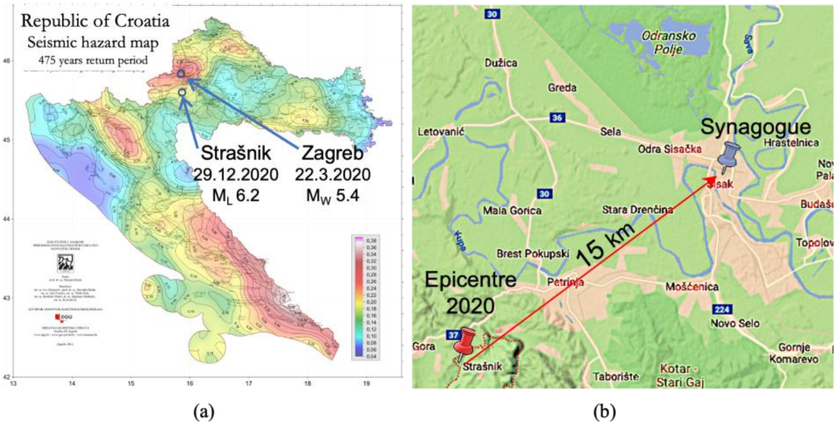


Fig. 1. Location of earthquakes in Croatia in year 2020 (a) and location of building of former Synagogue in Sisak, presented in this article (b)

Article presents and discusses the concept design of retrofitting and partial reconstruction of the monumental historic building affected by the 29. December 2020 earthquake having epicenter 15 km South-West of its location (see Fig.1(b). The concept is the result of teamwork of architects, conservators, and structural engineers (the authors of this article). The efficiency of proposed retrofitting measures has been assessed by program 3Muri which is in wider use for assessment of seismic resilience of masonry structures.

2 Description of building and concept of retrofitting

The building was constructed during the period from 1862 and 1892 in romantic historicism style as a synagogue financed by the local Jewish community of Sisak. During WW2 the building was expropriated and adapted for use by the authorities. From 1967 until 2020 earthquake, it serves as a music school. Due to its historic and architectural values it is registered as a cultural monument.

It is erected at very demanding location because of geological characteristics and archeological remains of Roman town Siscia established in the 1st century B.C. Beneath the building are layers of soft alluvial deposits up to 30m deep, formed by three nearby confluenting rivers: Sava, Kupa, and Odra (see Fig.1(b)). During the December 2020 earthquake was present phenomena of local soil liquefaction in wider area of town Sisak. The builders were aware of geotechnical conditions and thus foundations were constructed of brick masonry in good mortar down to depth of 280 cm.

Layout of building is of orthogonal shape long 20.15m and wide 16.15. The height of the building is 15,5 m. Parts of the building, as originally constructed, are of burned clay solid bricks laid in lime mortar. Brick dimensions are l/d/h=30/15/7.5cm. Their characteristic compression strength is 15 MPa. Lime mortar is of different quality in ground floor and first floor walls. Mortar on the first floor is of very poor quality with an estimated compressive strength of 0,2 MPa while the mortar of ground floor walls has estimated compressive strength of 0,7 MPa.

After expropriation of synagogue during the WW2, the ground floor interior was changed by adding partition walls of different thickness. The central open space was divided in height by massive new floor structures (see Fig.3(a)). Façade windows were partially or entirely closed by masonry infills. The massive masonry arches supporting the timber domes and domes themselves were preserved in the original form while passages between rooms were partially or entirely closed (see Fig.4).

(a) (b)

Fig. 2. Building before (a) and its front façade after (b) the earthquake of December 2020

Earthquake of 29. December 2020 seriously damaged masonry walls. Gamble of western façade collapsed and façade walls severely cracked (Fig.2 (b)). Other façade walls sustained less damages concentrated merely along the contacts of original and infilled parts of walls in the original openings. Similar patterns of damage developed in the first-floor walls (Fig.4 (a)) where masonry arches cracked along their perimeter lines. Lightweight timber dome (Fig.4 (b)) supported by masonry arches remained undamaged. In general, the western massive part of buildings sustained mayor structural damages. Assumably it may be the consequence of tilting of the western part of building due to local ground properties where phenomenon of liquefaction might develop during the earthquake excitation.

Learning from the earthquake response of building the concept of partial reconstruction and retrofitting was developed as joint endeavor of asset owner, architects, conservators, and structural engineers. The main suggestion of structural engineers was to use lightweight materials where possible to reduce future inertial horizontal forces due to earthquake excitation and to reduce vertical loading transferred to problematic ground.

(a) (b)

Fig. 3. Longitudinal (East-West) (a) and lateral (South-North)(b) cross-section of build-ingacquired from 3D point cloud model after earthquake of December 2020

(a) (b)

Fig. 4. The first floor (view towards West) damaged by the December 2020 earthquake (a) and attic with upper surface of the main timber dome (view towards West) (b)

3 Concept of partial reconstruction and retrofitting of building

Upon request of owner the layout building interior and shape and dimensions of façade openings should be returned to the original shape of building. It was a challenging task from the perspective of seismic resilience especially because of the conservator's request for preservation of original structural elements anywhere it is possible. However, the general understanding of the need to introduce the structural elements made of wood due to reduction of masses has been achieved. Following the suggestion of team members responsible for structural issues, intervention would encompass following:

- Removal of ground floor inner walls, floor structures above the ground floor and walls of the first floor including masonry arches, timber domes and roof structure.
- Restoring window openings on the ground floor to their original shape adding the r.c. encirclements.

- Repairing of ground floor façade walls by grouting and partial replacing of week mortar and construction of the inner r.c. frames (see Fig.5) and laying of horizontal r.c. tie beams atop the repaired ground floor façade walls.
- Construction of the first-floor inner masonry walls and façade walls extended to roof knee and gable walls with horizontal r.c. tie beams.
- Installation of laminated timber arches and domes on the first floor and CLT floor diaphragms (see Fig.6). Poz.1 and 2 are 15 cm thick CLT panel, Poz. 3 is 18 cm thick CLT panel, Poz. 4, 5 and 6 are timber domes.

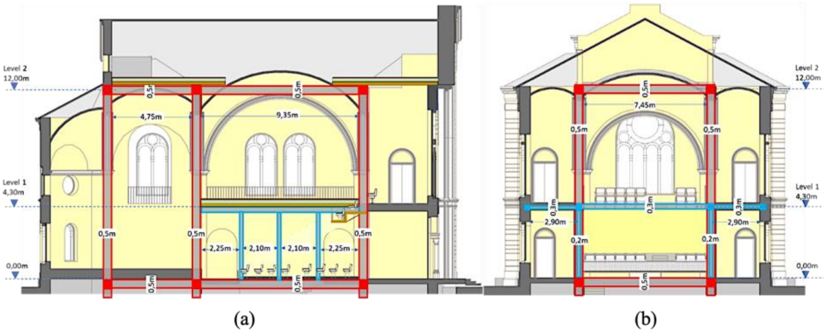


Fig. 5. Concept of retrofitted and added structural elements: longitudinal (East-West) (a) and lateral (South-North)(b) cross-section of building.

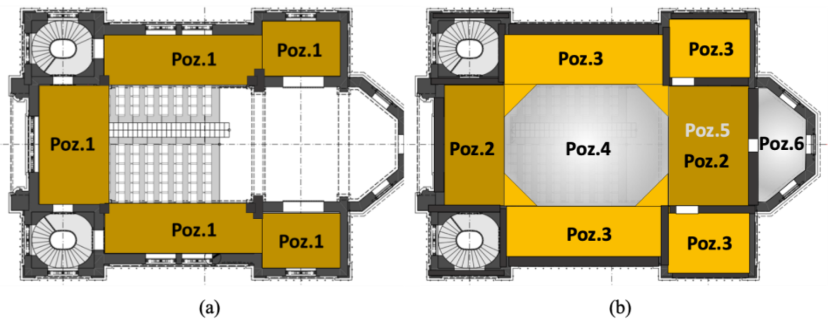
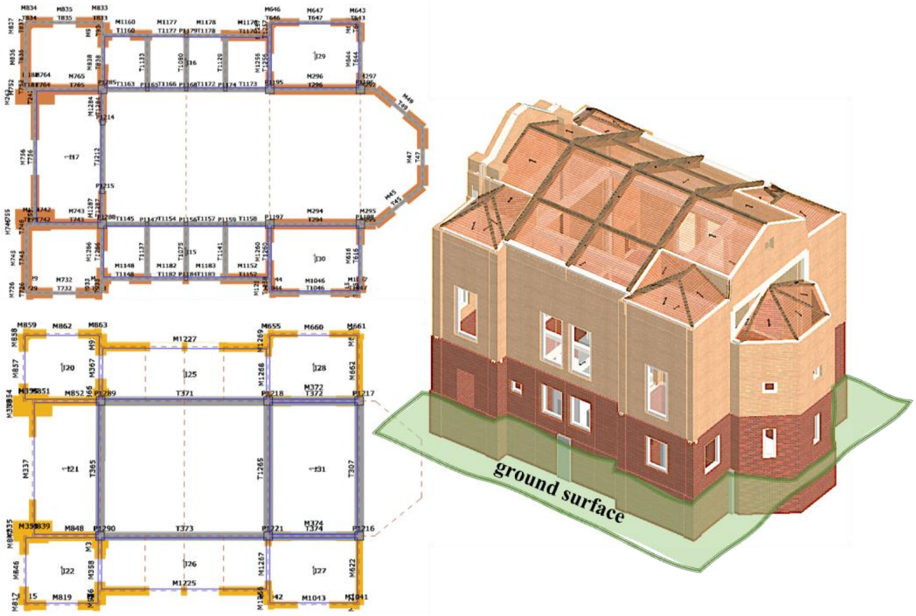


Fig. 6. Added horizontal diaphragms on first floor (a) and attic level (b)and timber domes on the attic level (b).

The thickness of the ground floor masonry walls is 61 and 46 cm, while the first-floor walls will be thick 46 and 30 cm. Use of the bricks of the same type as originally used is requested by conservators. The percentage of reinforcement of main inner frame columns is equal to 3.5% of their cross-section area. The percentage of reinforcement of main inner frame beams is equal to 1.2% of their cross-section area.

4 Seismic resilience assessment of the retrofitted building

The soundness and adequate earthquake resistance of proposed structural concept was verified by seismic analysis of the equivalent frame model based on the discretization in terms of piers and spandrels was created (SEM - Structural Elements Model). The simulation of earthquake response of structure was carried out by the 3Muri, v.13.9 [2] software for the assessment of structures constructed of masonry and mixed materials through a non-linear (pushover) and static analysis. Theoretical background and practical application of software is presented in [3]. 3Muri is also frequently used for assessment of buildings affected by the earthquakes in Croatia in the year 2020 [4] such is the herein presented case.

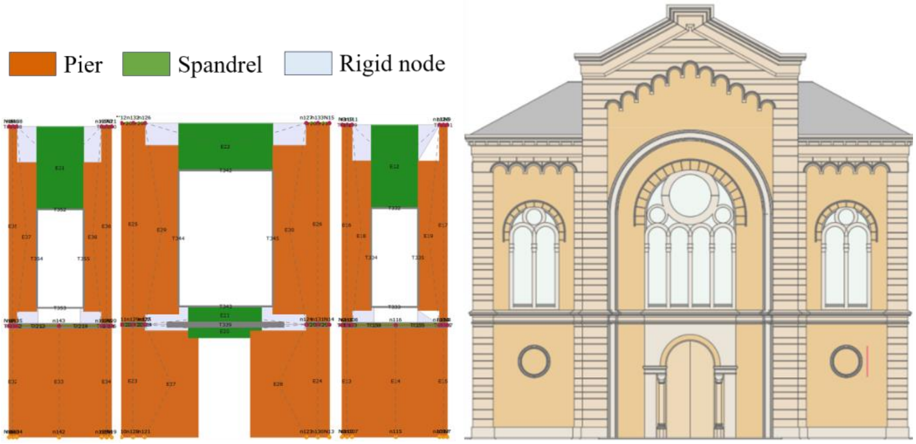


(a) (b)

Fig. 7. Ground floor and the first-floor layout of building model (a) and its 3D model (b)

In the presented layouts of ground and first floor (see Fig.7 (a)) and 3D model of building (see Fig.7 (b)) the position of existing and new structural elements (masonry walls, r.c. frames and timber roof structure). The existing masonry which will be repaired is in darker color while the new one is in lighter color.

In the equivalent frame approach, only the in-plane response of the URM walls is considered, and each wall is discretized by a set of masonry panels (piers and spandrels), where the non-linear response is concentrated, connected by a rigid area (nodes). The wall idealization into an equivalent frame affects both the elastic field, since it alters the actual deformability of the wall due to the simplification of introducing rigid nodes, and the nonlinear phase of the response, since the regions where the cracks and nonlinearity are likely to develop are assumed a priori. Despite these simplifications, this approach is one of the most spread both in engineering practice and at the research level thanks to its computational efficiency in performing nonlinear analyses and its reasonable accuracy, as proven by various numerical simulations in the literature [5]. Fig. 8 illustrates the discretization of the entrance façade walls.



(a) (b)

Fig. 8. Equivalent frame idealization (a) of the entrance façade (b)

In Table 1 are presented mechanical properties of masonry as used in the model of building. The properties of the existing masonry have been derived from the results of standard on-site testing of masonry ("flat jack" compressive test, mortar joint shear test) and laboratory testing of bricks and mortars. The properties of the new masonry were derived from the published sources. The sheer strength of masonry macro-elements was calculated according to Turnšek-Čačovič theory [6].

Table 1. Mechanical parameters assumed in the SEM model.

Solid brick and lime mortar[1]	**E [MPa]**	**G [MPa]**	**w [kN/m3]**	**f_m [N/cm2]**	**τ_0 [N/cm2]**
Existing masonry	1600	250	18	150	5
Newbuilt masonry	2100	350	18	230	7,6

(1) E: modulus of elasticity, G: shear modulus, w: average specific weights, f_m: compressive strength, τ_0: shear strength. The strength values must be divided by the CF, assumed equal to 1.2.

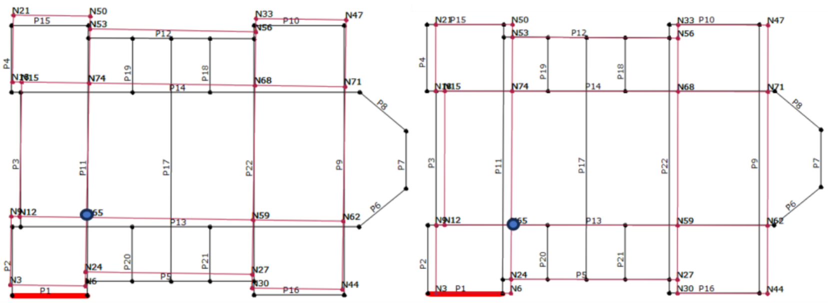
4.1 Results of the modal analysis

The modal analysis was performed to interpret the dynamic response of the whole building in the original and new configuration. The effect of replacing reinforce concrete heavy rigid floors with lightweight cross-laminated wooden floors is in reducing of total mass of building (M_{TOT}) for 9%.

Table 2 shows the main results of the modal analysis of building in the new, retrofitted and partially reconstructed configuration in terms of period (T), frequency (n), and percentage of participation mass (%Mx, %My, and %Mz). The results are illustrated by selecting the first 10 modes; the most significant ones in terms of participation mass are marked in grey.

Table 2. Results of the modal analysis

Lightweight rigid floors (CLT) M_{TOT} = 1.320.430 kg										
Mode	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
T [s]	0,67	0,47	0,45	0,43	0,33	0,25	0,25	0,23	0,22	0,19
n [Hz]	1,45	2,12	2,22	2,31	2,98	3,96	4,11	4,31	4,49	3,35
%M_x	0,00	0,05	0,11	**66,86**	4,71	0,02	0,80	0,90	0,00	1,15
%M_y	**60,88**	5,32	0,02	0,00	0,04	5,22	0,08	0,13	22,38	0,00
%M_z	0,00	0,00	0,02	0,00	0,00	0,00	0,00	0,00	0,00	0,00



(a) Mode 1 Control node (a) Mode 4

Fig. 9. Comparison of modal shapes for Modes 1 (a) and 4 (b)

4.2 Results of nonlinear static analysis

Nonlinear static analysis was performed on the global equivalent frame model (Fig. 7). Pushover curves were extracted by plotting the shear at the base of building (V) as a function of the mean displacement of the nodes placed at the last floor (d). The control node was defined according to displacements calculated by modal analysis (Fig. 8). The corresponding capacity curves ($V^* - d^*$) of the equivalent Single Degree of Freedom (SDOF) system were defined, by following the general principles of [7], based on the evaluation of the participation coefficient Γ and the mass M^* of each unit (having extracted from the 3D model the data related to each of them). Thus, each capacity curve was obtained by dividing the displacement d by Γ ($d^* = d/\Gamma$) and the base shear by the product ΓM^* ($V^* = V/(\Gamma M^*)$). Finally, for the seismic verification, the capacity curve was compared with the seismic demand. The analyses were performed by adopting, for each examined direction (+X, −X, +Y, and −Y), two different load patterns (LPs): proportional to masses (hereafter referred to as "uniform") and proportional to the product mass per height (hereafter referred to as "pseudo-triangular"). The results refer to the analysis step corresponding to a 20% decay of the base shear, assumed as representative of the Significant Damage limitstate (SD) and Damage Limitation limit state (DL).

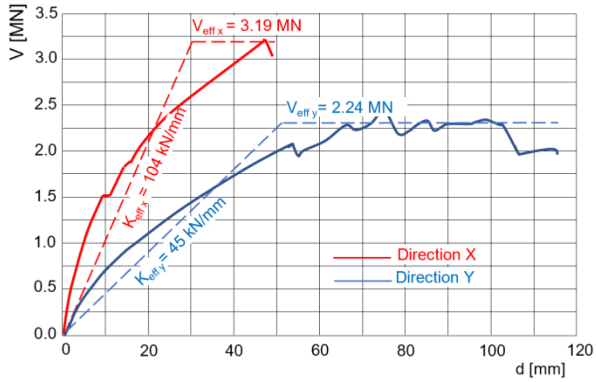


Fig. 10. Comparison of modal capacity curves in direction -X and +Y obtained by most significant analysis.

Altogether 24 analyses were performed and the capacity curves of the most significant ones, in -X and +Y are presented in the capacity curves in Fig. 9. The corresponding effective story stiffnesses (K_{eff}) and effective shear at the base of building (V_{eff}) were derived from the analysis results. As it can be seen from the values of periods (T) in table 2 and diagrams in Fig. 9 the stiffness and earthquake resistance of structure is much higher in the longitudinal direction (Y) due to the flexibility and ductility of the reinforced concrete frames creating the core of structure (Fig.7). The ductile mechanism of the structure response with sufficient level of resistance well justifies the selected strategy of structural strengthening introducing r.c. frames and lightweight, stiff CLT horizontal diaphragms and lightweight timber central dome.

The following tables 3 to 5 present the detail data of the response of structure. As it is presented, the repaired, strengthened and partially reconstructed building in each direction satisfies the demands of the Eurocode 8 (α parameters are higher than 1.0).

Table 3. Verification of structure seismic vulnerability

Limit state	PGA [m/s2]		α	
	Direction X	Direction Y	Direction X	Direction Y
Significant damage SD	1.062	1.713	4.721	7.613
Damage limitation DL	0.888	1.022	7.399	8.514

PGA: limit capacity acceleration, α=PGA/a_{gR}, a_{gR}: design ground acceleration at site

Table 4. The limit state parameters

Limit state	Direction X	Direction Y
SD	d_t = 7.7mm < d_m=36,6mm	d_t = 11.3mm < d_m=86,3mm
DL	S_d 3.5mm < d^*_y=26,2mm	S_d 5.4mm < d^*_y=46,1mm

d_t: the target displacement of the Multi Degree of Freedom (MDOF) system dt=d*t x Γ, d_m: the ultimate displacement of the Multi Degree of Freedom (MDOF), d^*_y: the yield displacement of the idealized SDOF

Table 5. Analysis parameters

Parameter	Direction X	Direction Y
Period of the equivalent system T*[s]	0.555	0.848
Mass of the equivalent system m*[t]	812.26	792.65
Total mass until achievement of a term of values w[t]	1537.38	1537.38
Ratio m*/w [%]	51.8	50.58
Modal participation factor Γ	1.17	1.12
Plasticization strength of the equivalent system F^*_y [MN]	2.73	2.01
Plasticization displacement of the equivalent system d^*_y [mm]	26.2	46.1
Ultimate displacement of the equivalent system d^*_u[mm]	41.8	103
Available ductility d^*_u/ d^*_y	1.60	2.23

In addition to seismic analysis, 3Muri program enables the static analysis. Load bearing cappacity of masonry is checked according to Eurocode 6. In herein describedcase, all walls and reinforced concrete element meets required cappacity as it is presented in Fig.11. In table below N_{ed} represents the design value of the applied axial force on the masonry pier, while N_{Rd}represents the resistance of the masonry pier to the applied force. All piers satisfy the requested criteria, since the ratio N_{ed} /N_{Rd}< 1.0.Parameter h_{ef} refers to the effective height of masonry pier and t_{ef} stands for the effective thickness of masonry element. The limit slendernes ratio h_{ef}/t_{ef}should not exceed the value of 20 for the unreinforced masonry. In our case the values are just slightly exceeded at some masonry piers.

Wall	Failed piers	Ned/NRd Max	hef/tef Max
1	0	0,45	15,10
2	0	0,69	15,10
3	0	0,62	20,26
4	0	0,68	15,10
5	0	0,45	15,10
6	0	0,15	7,05
7	0	0,16	7,05
8	0	0,15	7,05
9	0	0,36	20,26
10	0	0,38	20,26
11	0	0,48	12,62
12	0	0,46	15,10
13	0	0,66	15,10
14	0	0,65	15,10
15	0	0,46	15,10
16	0	0,38	20,26
22	0	0,40	20,26

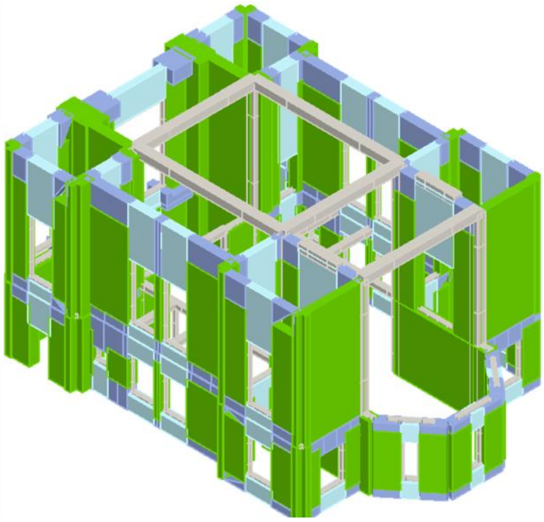
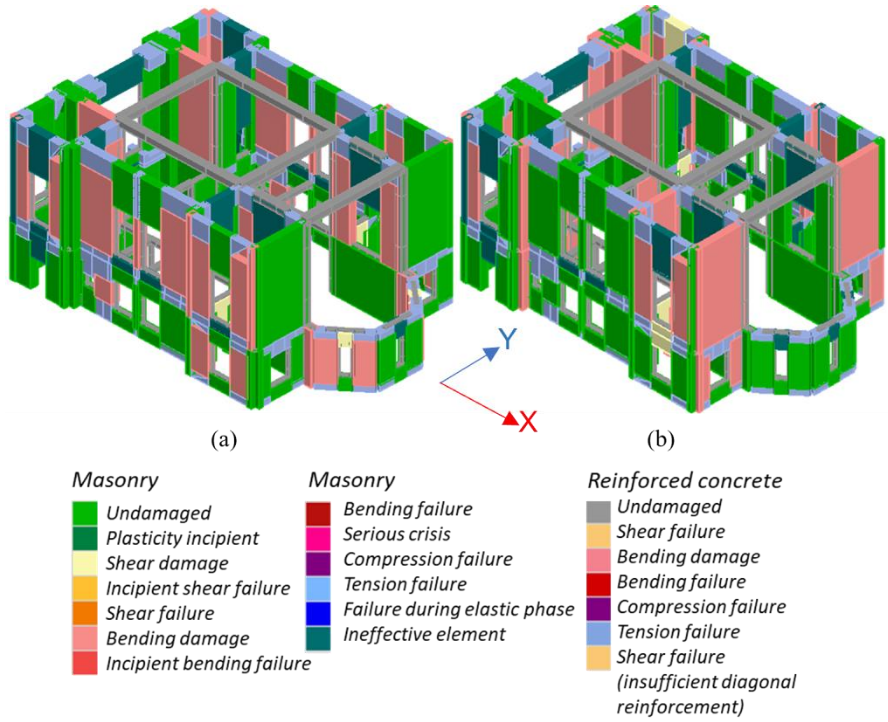


Fig. 11. Results of static analysis of structure

The distribution of damages at significant damage (SD) limit state due to action of earthquake action in longitudinal (X) and transversal (Y) direction is presented in Fig. 12 where the legend of diferent damage stages of structural elemnts is added. More the half of masonry piers and wals as well as all reinforced concrete columns and beams are undamaged. The rest of masonry pier suffered shear failures that did not jeopardize the stability and integrity of structure as whole. The global stability and integrity of the structure is provided by the strong inner reinforcedconcrete frames.



undamaged

Fig. 12. Damage pattern of structural elements resulted from the seismic analysis at achieved story displacement d of 48.8 mm and base shear force V of 3.20 MN in X direction (a) and story displacement d of 115.0 mm and base shear force V of 2.39 MN in Y direction (b).

The presented damage pattern of structural elements provides information on the critical parts of building and its global response to earthquake action. This information is crucial for the design of structure as whole because the weak parts can be identified and strengthened if needed. However, in the presented case there is no need for further strengthening of structural element because in configuration as presented it fulfills the requirements of Eurocode 8 as justified by data in Table 3 above.

5 Conclusion

Presented case of post-earthquake design of heavily damaged historic masonry building located in Sisak, Croatia describes the strategy and results of heritage building retrofitting design. Following the conservator's guidelines and owner's requirements for the future use of building the concept of retrofitting took into consideration the preservation of heritage character of building. Thus, the building envelope constructed of unreinforced masonry (URM) was preserved and reconstructed following the original form of building erected by the end of 19th century. In the inner part of structure is placed the strong reinforced concrete frame that together with URM provides sufficient earthquake resistance according to Eurocode 8. In order to reduce the masses all floor structures were constructed of cross-laminated timber instead of reinforced concrete what reduced the total mass of structure by 9%.

Acknowledgment

The presented case of earthquake resistance assessment of repaired, strengthened and partially reconstructed building is a part of design documentation produced by PLANETARIS Ltd, Vodnikova 11, 10000 Zagreb, Croatia. The authors of this article contributed earthquake resistance assessment, while other involved experts, coordinated by the director of PLANETARIS Ltd. Natko Bilić provided architectural content, data derived from the site investigations and conservator’s guidelines.

References

1. Prevolnik, S.:Analysis of accelerograms of 29. December 2020 Petrinja earthquake (in Croatian),University of Zagreb, Faculty of Science, Department of Geophysics, https://www.pmf.unizg.hr/geof/seizmoloska_sluzba/potresi_kod_petrinje_2020#, last accessed 2023/06/30.
2. 3Muri Homepage, https://www.3muri.com/en/, last accessed 2023/01/23.
3. Penna A.; Lagomarsino S.; Galasco A.: A nonlinear macro-element model for the seismic analysis of masonry buildings, Earthquake Engineering & Structural Dynamics, 43(2): 159-179.(2014)
4. Ademović, N., Toholj, M., Radonić, D., Casarin, F., Komesar, S., & Ugarković, K. (2022). Post-Earthquake Assessment and Strengthening of a Cultural-Heritage Residential Masonry Building after the 2020 Zagreb Earthquake. Buildings 2022, 12(11), 2024; https://doi.org/10.3390/buildings12112024, last accessed 2023/06/30.
5. Marino, S.; Cattari, S.; Lagomarsino, S.; Dizhur, D.; Ingham, J.M. Post-Earthquake Damage Simulation of Two Colonial Unreinforced Clay Brick Masonry Buildings Using the Equivalent Frame Approach. Structures 2019, 19, 212–226.
6. Turnšek, V.; Čačovič, F. Some experimental results on the strength of brick masonry walls. In: Proceedings of the 2nd international brick masonry conference. Stoke-on-Trent, UK: British Ceramic Research Association, p.149-156.(1971)
7. Fajfar, P. Capacity spectrum method based on inelastic demand spectra. Earth Eng. Struct. Dyn. 1999, 28, 979–993.