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Case-study of a typical residential building in the Lower town district of the city of Zagreb, Croatia

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Abstract

The paper presents a case-study of a typical residential building in the district Lower town in Zagreb historical center. The building is the representative example of unreinforced masonry buildings that were built at the end of nineteenth and at the beginning of twentieth century. Many buildings of this typology, as well as the case-study building, suffered moderate to severe damage due to ML5.5 earthquake that struck Zagreb in March 2020. Due to large number of these buildings, it is unrealistic to expect their quick replacement with new buildings or even higher investments in retrofitting to achieve the level of seismic performance prescribed by today's standards. Therefore, the modifications were made to Croatian Technical Regulation for Building Structures concerning seismic requirements. Depending on the building use, a certain level of safety needs to be fulfilled. There are in total four levels and for the residential buildings, the regulations propose the so-called Level 2 seismic requirements. Results of numerical analyses based on the pushover method which are performed to obtain seismic capacity level that is validated with the proposed demand, are presented in this paper. The seismic capacity of the building is discussed and necessary strengthening measures for this building typology are recommended.

Key words: unreinforced masonry, nonlinear modelling, pushover method, retrofitting measures

1 Introduction

In the Lower town, the district in the historic center of the city of Zagreb, the capital of Croatia, most of the buildings are unreinforced masonry buildings (URM) built in blocks at the end of the nineteenth and the beginning of the twentieth century. Building heights are usually between 15 and 25 meters, having 3 to 4 storeys. In the earthquake of March 22nd, 2020 the most frequent damage that occurred in this typology of buildings involves collapse of chimneys, gable walls at attic level, and other cantilever parts at the top of the building (parapet walls, various cantilevers, etc.), and damage to roof structure [1]. Furthermore, separation of walls along the height caused detachment of floor structures, and extraction of joists from their supports. Many buildings were affected by formation of diagonal cracks in load-bearing (structural) and non-bearing (non-structural) walls and lintels due to exceedance of in-plane load-bearing capacity.

The survey of earthquake damage to buildings in Zagreb according to the procedure described in [2] showed that comprehensive retrofitting or replacement of the affected buildings with the new ones would be hard to achieve and that substantial investments were required to ensure the modern standards of seismic resistance. Therefore, to ensure that reconstruction measures will lead to a certain level of seismic resistance of old URM buildings, the Croatian Technical Regulation for Building Structures (CTRBS) has prescribed levels of seismic performance requirements depending on the building use, which may be applied only to buildings subjected to post-earthquake reconstruction. Although the Croatian standard for seismic assessment and retrofitting of buildings HRN EN 1998–3 requires two limit states (significant damage, SD, and damage limitation, DL), the CTRBS defines only one limit state - significant damage. The levels defined by the regulation roughly correspond to the following safety indices: for level 2 the safety index is approximately 0.5, for level 3 the safety index is about 0.75, while for level 4 it is 1.0.

Level 2: Building structural repairs and local strengthening measures, necessary to achieve mechanical resistance and stability of the building for seismic action associated with a reference probability of exceedance of 10 % in 10 years (return period of 95 years), for the limit state of significant damage.

Level 3: Retrofit of the building structure using methods and measures which ensure mechanical resistance and stability of the building for seismic action associated with a reference probability of exceedance of 20 % in 50 years (return period of 225 years), for the limit state of significant damage.

Level 4: Building mechanical resistance and stability needs to be in accordance with the relevant seismic standards of the HRN EN 1998 series. Retrofit of the building structure using methods and measures which ensure mechanical resistance and stability of the building for seismic action associated with a reference probability of exceedance of 10 % in 50 years (return period 475 years).

This paper presents a detailed analysis of a URM residential building. Significant damage limit state is computed by pushover analysis using different seismic force patterns. The analysis is carried out by considering the seismic retrofitting measures that are necessary to achieve Level 2 seismic performance according to CTRBS.

2 Description of the numerical model and analysis methods

The case-study building shown in Figure 1 is the representative example of the building typology in the Lower town in Zagreb. The building was built in 1922 and has a basement, ground floor, 3 storeys, and an attic. Plan dimensions are 24.4 × 12 m on the street side and 10.6 × 12 m in the courtyard, with the total gross plan area of approximately 407 m^2. The building height is 22.7 m. Annexes are connected with the main part of the building that is oriented to the street. The structure satisfies the criterion of regularity in height, while the criterion of regularity in the floor plan is not fully met. The floor plan of the building is irregular, with heterogeneous floor systems that do not have the required properties of rigid diaphragms, so such an overall condition is particularly unfavorable and susceptible to undesirable response to earthquakes. The external and internal load-bearing walls are made of solid bricks of the old format (290 × 140 × 65 mm) with a thickness of 90, 60, 45, 30 and 15 cm. The partition walls are 7 and 15 cm thick, and also built with solid bricks. The walls are interconnected by lintels, parapets and beams of which composition and quality are not fully known. The floor system above the basement at the street side of the building is a reinforced concrete slab with a system of reinforced concrete beams (otherwise, masonry vaults are often used instead of slabs for this building typology), and in the above stories there are timber joists oriented in transverse directions with a rubble filling inside the floor structure. The staircase is in the central part of the building between two annexes.

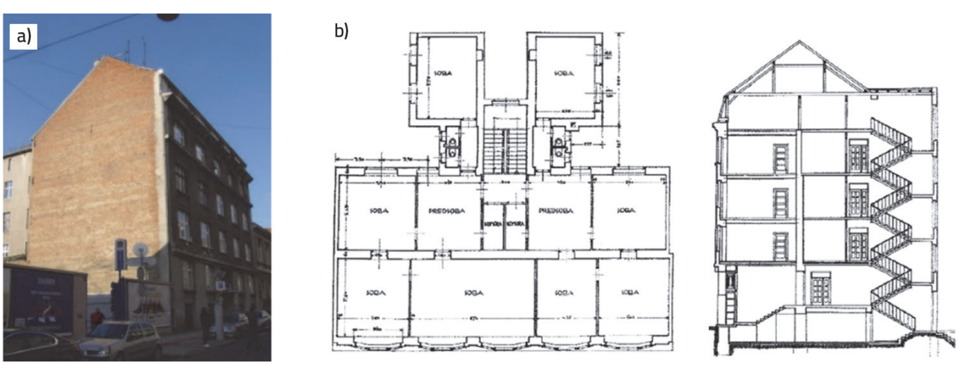

Figure 1. a) Photograph of the building; b) Archive architectural drawings.

Numerical model shown in Figure 2 is created in the software package based on the finite element method Etabs v17. [3], specialized for the design of buildings in seismically active areas. Program allows for adopting material characteristics and user definition of element capacity curves, so it can be efficiently used to model nonlinear behavior of brick or stone masonry buildings [4]. In the model, the cracking of cross sections that occurs during an earthquake is taken into account according to Croatian standards HRN EN 1998–1 and HRN EN 1998–3, and American guidelines for seismic assessment of existing buildings ASCE / SEI 41–13, ASCE / SEI 41–17 and ATC-40. According to these regulations the bending stiffness of reinforced concrete beams was reduced to 30 % of the initial stiffness, while for columns and walls this stiffness was reduced to 50 % of the initial stiffness. The shear stiffness of all elements was taken with 50 % of the initial stiffness. The nonlinear behavior of the elements was taken into account using plastic hinges at critical points in the structure. The load-bearing capacity of walls and lintels is analyzed for different failure mechanisms and dominant type of failure is selected for the definition of load-bearing wall capacity curve. Shear failure that is induced by the development of diagonal cracks in the wall or shear failure by wall sliding has been shown to be dominant. Wall failure initiated by material crushing at the bottom of the element is also present. The rigid diaphragms assumption is assigned to the floor structures. Hence, this is one of the main properties that needs to be ensured by retrofitting measures in a set of interventions related to the post-earthquake building repairs.

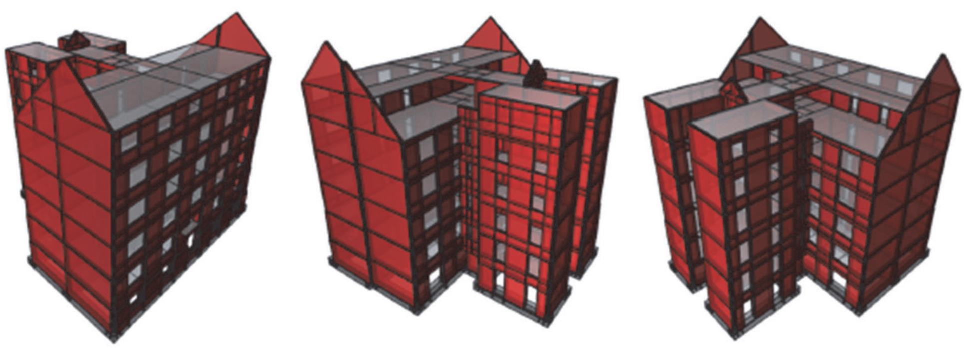

Figure 2. Numerical model of the building [3]

The pushover analysis is carried out by using the target displacement of mass center (CM) (Figure 3.a). Six different patterns of lateral force distribution that are displayed in Figure 3.b are applied to the model. For each pattern of the load, both directions with positive and negative sign are considered, and for each direction, a centric action and additional two with the eccentricity value of ± 5 % of perpendicular plan dimension are defined. A total of 72 calculations were carried out, but the results will be presented only for the relevant cases.

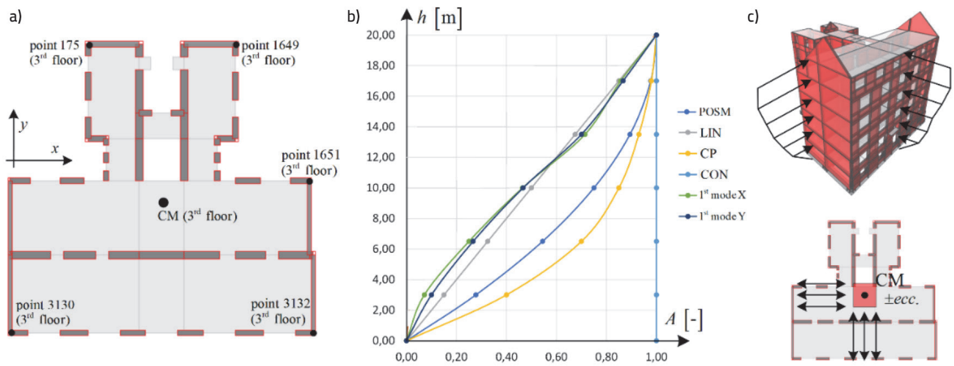

Figure 3. a) Building plan with the target point; b) Lateral force patterns; c) Schematic view of the lateral force application to the building model

3 Analysis results

The analysis revealed which patterns of lateral load distribution are the most unfavorable for the building response. It has been shown that the same patterns are relevant for both directions of loading (with a somewhat smaller or larger difference), and due to the high extent of the results only the most significant ones are presented below. The linear distribution (LIN) proved to be the most significant. It approximately corresponds to the distribution of horizontal forces according to the lateral force analysis method. A similar response of the structure, but somewhat more favorable, is for the distribution of the load according to the shear building deformation line, which is a parabola (CP). The most favorable response and the maximum load-bearing capacity of the building structure was obtained for the lateral load that is proportional to the inertial forces caused by the constant acceleration of the masses (CON). This pattern, along with the first vibration mode in that direction, is obligatory for the control of computation when doing pushover analysis. The pattern that is proportional to the mode that has significant mass participation in a certain direction (1st mode_X, 1st mode_Y) proved to be almost equal to the linear distribution. The results for these patterns are not shown in the final diagrams.

3.1 Building capacity curves for direction X

Figure 4 shows the capacity curves of the building for X direction of the lateral force. Only the most significant curves are shown and marked, and those curves obtained for the load pattern according to the force distribution by the method of lateral forces (green curves X (Y) _P) and the linear shape of the load (blue curves X (Y) _P_lin) are shown to be relevant. The number in front of the mark indicates different directions and eccentricity of the load. For the purpose of a qualitative presentation of the failure mechanism and critical elements in the structure, significant points on the curve are marked and the state of the structure for this level of lateral load is shown.

It can be noticed in Figure 4.a that several structural elements reach DL limit state, which is manifested on the curve as a decrease in stiffness for the lateral force value of 2000 kN (base shear - BS = 6.2 %) and displacement of the CM on the 3rd floor of 1.5 cm. The first failures of structural elements occur at a value of lateral force of 3200 kN (B.S. = 10 %) and the 3rd floor CM displacement of 3.1 cm. It can be observed that the several lintels are in the SD limit state. The next significant point marks the failure of the lintels on the street facade of the building for 3700 kN force value and the 3rd floor CM displacement of 4.2 cm. At this point in the curve, all the walls are still in a DL limit state. The first wall failure occurs at a force value of 4000 kN (B.S. = 12.5 %) and the 3rd floor CM displacement of 5.1 cm. These are the walls on the 3rd floor of the central longitudinal axis of the building. Consequently, local failure of the floor structure may occur. It may even be stated that the building is in a near collapse (NC) limit state and it may be classified with the highest degree of damage. However, even in the event of a local failure, the rest of the structure is unlikely to collapse for this load level. The next key point marks the exceedance of the SD limit state due to failure of the walls on the lower floors. Critical elements are marked by red color in the Figure 4.a. Global failure occurs at a lateral force value of 4200 kN (B.S. = 13 %) and the 3rd floor CM displacement of 6.3 cm. In addition to the elements in the central axis, the walls of the higher floors on the courtyard façade (west wall) also reached SD limit state.

Figure 4.b shows the capacity curves for the negative direction X of the lateral load. Similar as in the previous case, the lintels that first reach the SD limit state, proved to be critical. It can be observed that in this case the failure mechanism of the load-bearing system begins with a failure of the west courtyard wall on the 1st and 2nd floor. They reach the SD limit state at the level of the lateral force of 3900 kN and the 3rd floor CM displacement of 5.1 cm. The final failure of these walls occur at a displacement of 5.6 cm. These walls are relatively rigid and are located far from the floor CM, and they are affected by a significant lateral force due to torsional effects caused by asymmetry and eccentricity of the load.

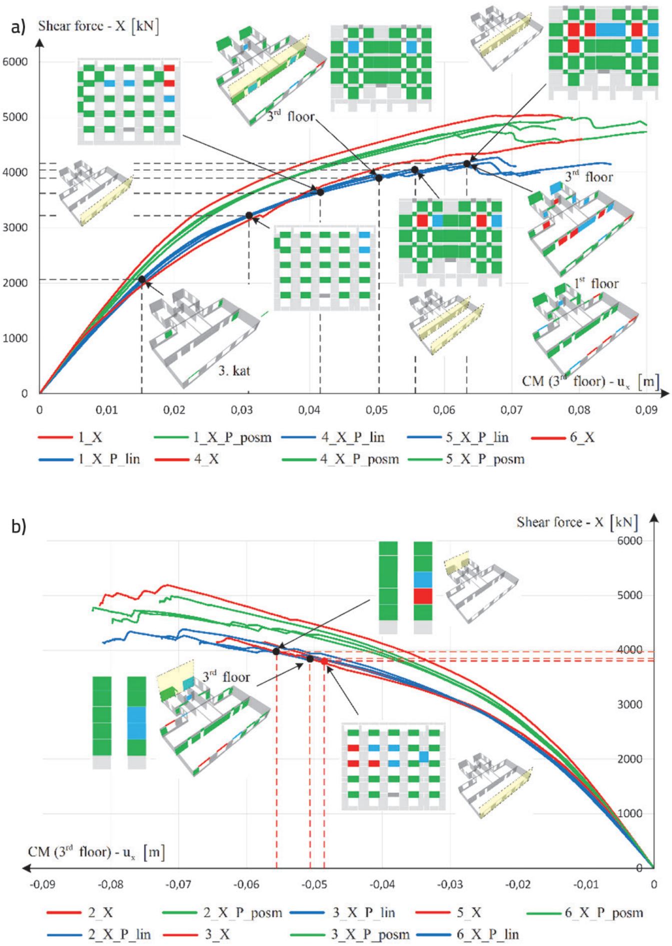

Figure 4. Capacity curves for direction X of lateral load

3.2 Building capacity curves for direction Y

Figure 5.a shows the capacity curves of the structure for the Y direction of lateral force. It can be observed that certain structural elements reach the DL limit state at a force value of 2600 kN (B.S. = 8.1 %) and a displacement of the 3rd floor CM of 1.6 cm. These elements are mostly located on the 3rd floor. Similar as in the previous load case, for Y direction of load application there is a regular occurrence of exceedance of the SD limit state for lintels, which requires engineering attention and a plan of their strengthening. The critical lintels that have a limited rotation capacity are located at the edge of the central transverse walls along the staircase towards the west side of the building. The onset of failure of structural elements occurs at a force of 4500 kN (B.S. = 14 %) and the 3rd floor CM displacement of 4.3 cm. The critical elements that first reach SD limit

state are the gable walls of the 1st floor on the north side of the building, followed by the gable walls on the 2nd floor. As the load increases further, the base shear force of the building continues to increase regardless of the wall failure. The next elements in which the SD limit state is exceeded are the courtyard facade walls in the transverse direction on the 2nd and 3rd floor. As for direction X, the critical wall failures may be observed on the higher floors of the building. It should be noted that the limit state verification is carried out at the level of the whole structure and at the level of the element. If an element fails before the global load-bearing reserves are utilized, that element should be strengthened and ensured with a sufficient deformation capacity. Ultimate 3rd floor CM displacement when a global building collapse occurs is approximately 6.0 cm.

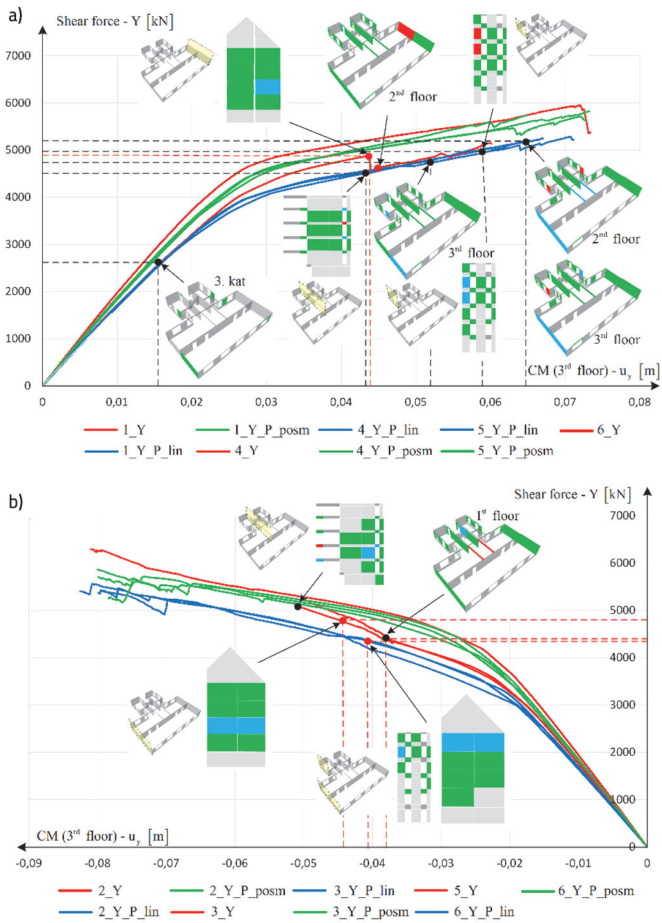

Figure 5. Capacity curves for direction Y of lateral load

Capacity curves of the structure for the negative Y direction (Figure 5.b) of the lateral load also show that the lintels, which first reach the SD limit state, proved to be critical. In addition to the gable walls, the courtyard part of the staircase wall on the 1st floor of the building also proved to be critical for this load direction. The failure mechanism of the vertical load-bearing system begins with the south gable wall on the 3rd floor or in the western part when it is initiated by the damage of staircase wall on the 1st floor of the building. They reach a SD limit state at the level of lateral force of approximately 4300 kN and the 3rd floor CM displacement of 3.9 cm. This is followed by the failure of these walls. The gable walls are relatively rigid and are located far from the floor CM, and due to torsional effects caused by asymmetry and eccentricity of the load, they carry significant shear force.

3.3 Capacity and building deformation requirement

The building capacity curves reduced to an equivalent single degree of freedom (SDOF) system are presented below. The procedure is performed according to the N2 method [5, 6]. For each of the 72 load cases, capacity curves were determined and the procedure of verification to the peak ground acceleration is made. However, due to high extent of the results, only the relevant curves and important parameters are presented.
Figure 7. Capacity curve and displacement requirement of an equivalent SDOF system
Diagrams in Figures 7.a and b show idealized capacity curves of an equivalent SDOF system for ground acceleration of 0.125 g on a bedrock which corresponds to a return period of 95 years according to the CTRBS Level 2 requirement for residential buildings. The results for the X and Y directions for the case of a linear lateral force pattern and without eccentricity are presented. Important parameters are marked, and it can be concluded that the deformation capacity of the system is higher than demand for both directions and that there are some load-bearing reserves. If we were to carry out the same procedure by increasing the ground acceleration, a maximum ground acceleration that would satisfy the displacement requirement can be determined. In that case, the maximum ground acceleration of approximately 0.17 g for the X direction and 0.16 g for the Y direction is obtained.
If eccentricity is considered, the capacity of the building drops significantly. Diagram in Fig. 7.c shows an idealized capacity curve of SDOF system for peak ground acceleration of 0.125 g considering X direction of lateral force distribution (X_P) with the presence of an eccentricity of 5 % of the building plan dimension. The deformation capacity of the system is approximately the same as the demand and for this case the system has no additional load capacity. The peak ground acceleration that satisfies the displacement requirement is approximately 0.125g, and this load case has proven to be most relevant. Diagram in Fig. 7.d shows an idealized capacity curve of an equivalent SDOF system ground acceleration of 0.125 g and Y direction of lateral load distribution according to the lateral force method (Y_P) with the presence of an eccentricity of 5 % of the building length. Here again, it can be concluded that the deformation capacity of

the system is approximately the same as the demand and the system has no additional load capacity. Finally, it can be stated that the peak ground acceleration on the bedrock at which the displacement requirement is met is approximately 0.125 g for both directions, provided that the design assumptions are met and the capacity according to the load capacity curve of the system in the relevant load case is enabled. This implies the strengthening of individual elements that should ensure a certain level of SD limit state before reaching the maximum load-bearing capacity of the building.

4 Necessary retrofitting measures and concluding remarks

The case-study building was significantly damaged in the Zagreb earthquake and in order to meet the necessary assumptions and constraints on which the global building response is based, it is necessary to carry out repairs and implement certain strengthening measures to reach the Level 2 seismic performance requirement according to CTRBS. The summary of the seismic retrofitting measures includes the following:

- Floor structures need to be strengthened and properly anchored to the walls in order to perform as rigid or semi-rigid diaphragms. This is one of the crucial measures because diaphragms ensure box-like behavior of the building, even distribution of the seismic load to the walls and prevent formation of the local out-of-plane failures of walls. It is especially important to provide a connection with the gable walls.
- Continuous connection of all perpendicular walls needs to be ensured.
- Targeted repair and strengthening of all load-bearing elements damaged in the earthquake should be performed. Elements need to be repaired to their original load-bearing capacity.
- Targeted strengthening of critical walls should be carried out.
- Systematic retrofit of lintels that represent a weak links in the load chain should be carried out. Reinforcement of the lintels is recommended in order to prevent their complete disintegration, which could endanger human lives. The purpose of the intervention on lintels is to ensure compactness during an earthquake. Strengthening may be performed using reinforced concrete, FRP materials, or adding new structural elements where possible.
- In addition to the strengthening of the primary seismic elements, it is important to consider the secondary elements (non-structural walls) that need to be properly connected to the load-bearing structure to ensure its local stability.

Finally, the recommended strengthening measures should improve the seismic performance of the building that was presented herein. The decision on the type and extent of the retrofitting should also take into account socio-economic aspects, such as intervention costs, the importance of the building, the usability of the building during the reconstruction period, etc.

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