



Performance-based seismic assessment of a typical unreinforced masonry building in Zagreb center

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Abstract

Seismic performance assessment of a masonry building in Zagreb, the capital of Republic of Croatia, is presented in this paper. The building is a representative example of a typical unreinforced masonry (URM) building in the Zagreb historic center. The building was built in 1920 what was before the first technical regulations considering seismic action were adopted in the country. Thus, it belongs to a typology of buildings without the necessary seismic resistance. In March 2020, a moderate ML5.5 earthquake struck Zagreb in which the analyzed building suffered severe damage. The building survey was performed on the day of the earthquake and the building was categorized as temporarily unusable. Severe damage to the partition walls and slight to moderate damage to the load-bearing walls was observed, and there was a partial collapse of gable walls due to formation of out-of-plane mechanisms.

Assessing the seismic behavior of existing unreinforced masonry buildings is a particular challenge for researchers but also for the engineers who ultimately carry out retrofit of URM buildings. In practice, there is a ubiquitous discussion about which numerical methods are applicable and reliable enough to perform seismic performance assessment of existing buildings. Often due to the very limited resources and available time, seismic assessment based on numerical models for ordinary buildings that have a basic value of importance factor, is limited to a use of linear methods and nonlinear static procedures.

For the building presented in the paper, firstly the out-of-plane wall failure mechanisms are analyzed in order to determine the necessary assumptions for the analysis of the global building response. This form of the wall failure can significantly endanger the overall vertical load-bearing capacity and stability of the structure. Further, it is important to highlight that very few countries have technical regulations that define procedures for the assessment of this form of structural failures. Out-of-plane mechanisms have been identified for this building, and the results are given in parametric form that considers different wall height and thickness. Moreover, for walls that are connected to transversal walls the effect of friction is also included. Based on this analysis, necessary assumptions for the numerical calculation of the global model are presented. A global structural response is determined firstly by a linear analysis carried out using the response spectrum which is a common method for design of a new structure. In this model, the assumption of the linear elastic behavior of the structure with cracked cross sections is used. Next, the nonlinear numerical computation using pushover method is performed. Material nonlinearity is taken into account, while geometric nonlinearity is not considered due to the high stiffness of the building and small displacements before failure.

Finally, a discussion about discrepancies that are present in the parameters of numerical model is presented, emphasizing the unreliability of the model, which cannot be fully eliminated, but should be accounted for when interpreting the results, and drawing the conclusions that are based on numerical models.

Keywords: unreinforced masonry, out-of-plane mechanisms, nonlinear modeling, pushover method



1. Introduction

The earthquake that struck the city of Zagreb on March 22, 2020 damaged many unreinforced masonry buildings, most of which were constructed in the late 19th and early 20th centuries within masonry building aggregates. This moderately strong earthquake, with a magnitude of 5.5 and an epicenter about 10 km from the city center, caused significant property damage. Most of the damage to the buildings refers to the collapse of chimneys, gable walls in the attic and other projecting parts in the upper part of the building (parapet walls, various projections, etc.), as well as damage to the roof structure [1]. In addition, the separation of walls along the height caused the detachment of floor structures and the pulling out of beams from their supports. Many buildings were affected by the formation of diagonal cracks in load-bearing (structural) and non-load-bearing (non-structural) walls and lintels due to exceeding the load-bearing capacity. Moreover, an aggravating circumstance for the City of Zagreb itself is the large population in the affected area and a relatively large number of old buildings of similar typology that do not even meet the minimum seismic requirements given the construction period. It is anticipated that replacement of severely damaged buildings and/or extensive retrofitting to improve the seismic performance of existing buildings will be very challenging, time consuming and require significant financial resources. It should be stressed that Croatia is in a particularly unfavorable situation as there was another 6.3 magnitude earthquake in the town of Petrinja (only 50 km from Zagreb) in December 2020 and the country, like many others, is in crisis due to a Covid 19 pandemic.

The building analyzed in this paper is a representative example of the most common building typology in the center of Zagreb. Existing buildings are not expected to meet current seismic performance standards, and the earthquake that occurred in March 2020 reminded many of the high seismic vulnerability of the Croatian capital. The issue of the importance of assessing the seismic performance of buildings built before the introduction of the first earthquake guidelines is highlighted. Therefore, the assessment of the seismic behavior of conventional buildings must be simple, fast and accessible to a larger number of civil engineers. Therefore, different complexity analyzes are presented for the case study building, consisting of a kinematic local analysis, and linear spectral analysis and nonlinear pushover analysis that are used to assess the global building behavior. Relevant conclusions on building behavior are highlighted and recommendations for seismic retrofitting are presented.

2. Analysis methods

One of the most frequently observed seismic vulnerabilities of buildings in the center of Zagreb is the occurrence of local mechanisms known as out-of-plane wall failure. When wall shaking occurs, it poses a major threat to structural stability and human life. In recent years, extensive research has been conducted on out-of-plane wall failure [2,3]. Some countries, such as Italy, have gone a step further and have included verification procedures for this type of mechanism in technical guidelines [4]. In this paper, a linear kinematic method is used for out-of-plane wall failure as a first step to evaluate the seismic behavior of the building. The method assumes the formation of local mechanisms with a rigid block and the formation of line joints at the edges of the elements. For walls where this was possible, the effect of friction at the joint with the transverse walls was considered. The kinematic chain of the blocks is defined as a single degree of freedom (1DOF) system. After defining the geometry of the kinematic chain and the boundary conditions, it is important to consider all the forces acting on it, including the inertial force caused by the acceleration of the block. The activation coefficient α_0 of the mechanism is determined by applying the equation of virtual work:

$$\alpha_0 \left(\sum_{i=1}^n P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j} \right) - \sum_{i=1}^n P_i \delta_{y,i} - \sum_{h=1}^o F_h \delta_h = L_{fi}. \quad (1)$$

In equation (1), P_i are the weights of the blocks involved in the mechanism, P_j are the forces transmitted to a particular block, F_h is the general horizontal force acting on the block, $\delta_{x,i}$ is the horizontal virtual



displacement, and $\delta_{y,i}$ is the vertical virtual displacement. L_{fi} is the virtual work of the internal forces (e.g. friction, tensile forces). The coefficient α_0 multiplies all forces induced by the block inertia. To determine the acceleration that activates the mechanism, the effective mass and effective mass ratio e^* of the 1DOF system must be determined:

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{px,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{px,i}^2}; \quad e^* = \frac{g M^*}{\sum_{i=1}^{n+m} P_i}. \quad (2)$$

Finally, the spectral acceleration without considering a reduction factor of the 1DOF system activating the mechanism in motion is given by $a_0^* = \alpha_0 g / e^*$.

Next, the analysis of the seismic behavior of the global model was performed using response spectrum method, which is a common procedure for designing new buildings. The assumption is the linear elastic response of the structure with the use of cracked cross sections. The method is applicable to buildings regular in height without any restrictions as it considers any number of vibration modes, thus satisfying the criterion for activating 90% of the mass of the structure. The behavior factor was chosen to be $q = 1.5$, since the mechanisms of building failure and the possibility of energy consumption, as well as the ductility of the building are not known. The calculation was performed for values of peak horizontal ground accelerations corresponding to a return period of 95 and 475 years for limit state of significant damage (SD) which is in accordance with the technical regulation for structures in Croatia for reconstruction after the earthquake.

Finally, a nonlinear static calculation was performed using the pushover method, which takes into account the material nonlinearity and the general nonlinear behavior of the structure for a given horizontal load. The method was performed on a three-dimensional model individually for each horizontal direction. Different forms of lateral load patterns were applied and their influence on the load carrying capacity of the building was analyzed. Eccentricities in the force application were also considered. The numerical calculation was carried out with the displacement control. The material nonlinearity was considered at the points of formation of plastic hinges, while the geometric nonlinearity was not considered, since its influence is small due to the high stiffness of the building and the small displacements before failure. In nonlinear structural calculations and in the definition of capacity curves with load reduction.

3. Building parameters and analysis results

The case study building that is shown in Figure 1 was built in 1922 and has six floors: basement, ground floor, 3 storeys, and an attic. The floor plan is irregular and consists of a main building part oriented to the street side and two annexes on the courtyard side. The staircase is located in the central part of the building between the two annexes. The floor plan dimensions are 24.4×12 m on the street side and 10.6×12 m on the courtyard side, with a total gross floor area of approximately 407 m^2 . The floor system above the basement on 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 in this building typology), and on the floors above there are transversely oriented timber joists with rubble infill within the floor structure. The external and internal load-bearing walls are constructed of solid brick of the old format ($290 \times 140 \times 65$ mm) with thicknesses of 90, 60, 45, 30 and 15 cm. The parameters used for the masonry walls are: volumetric weight of masonry $\gamma = 18 \text{ kN/m}^3$, modulus of elasticity $E_M = 1500 \text{ N/mm}^2$, shear modulus $G_M = 500 \text{ N/mm}^2$, compressive strength $f_m = 3.4 \text{ N/mm}^2$, initial shear strength $f_{v0} = 0.160 \text{ N/mm}^2$, and friction angle of $\mu = 0.4$. The walls are connected by lintels, parapets and beams whose composition and quality are not fully known.

To evaluate the kinematic behavior of two gable walls, the procedure based on the assumption of a rigid block and the method of virtual work is carried out using Wolfram Mathematica v.12.1. For the evaluation of the global seismic performance of the building, the numerical model is created in a software package Etabs v17. based on the finite element method. In the model, the material properties of the brick material are adopted and element capacity curves corresponding to the nonlinear behavior of masonry walls are defined to perform nonlinear analysis [5]. In the model, the cracking of the cross-sections that occurred during the



earthquake is considered according to the Croatian standards HRN EN 1998-1, HRN EN 1998-3 that are adopted Eurocode 1998, and the American regulations for seismic assessment of existing buildings ASCE / SEI 41-13, ASCE / SEI 41-17, and ATC-40. According to these regulations, the flexural stiffness of reinforced concrete beams was reduced to 30% of the initial stiffness, while for columns and walls it was reduced to 50% of the initial stiffness. The shear stiffness of all elements was taken as 50% of the initial stiffness. The nonlinear behavior of the elements was accounted for by the local opening of the plastic hinges at critical points in the structure. The bearing capacity of walls and lintels is analyzed for different failure mechanisms and the dominant failure mode is selected for defining the bearing capacity curve of the wall. Shear failure induced by the development of diagonal cracks in the wall or shear failure induced by wall sliding is found to be dominant. Wall failure induced by material crushing at the bottom of the element is also present. Floor systems are associated with the adoption of rigid diaphragms. This is one of the main features that must be ensured by retrofit measures in a set of interventions related to building repair after the earthquake.



Fig. 1 – Photographs of the building after the earthquake: a) aerial view, b) south façade; c) typical storey floor plan

3.1 Activation of out-of-plane mechanisms

Parametric kinematic analysis was performed for two walls, shown in Figure 2. The mechanisms were modelled as a single rigid block, since there are no reinforcing elements in the levels of floors that could lead to two body motion. The plan of virtual displacement is determined for the unit value of control node displacement on the top of the wall and segments of the wall are marked. The figure shows the initial position of the line joint at the wall base. In the parametric analysis, the line node is displaced towards the top of the structure since it is known that the position of the formation of the node itself is not fully known. The friction at the connection of wall Z1 with perpendicular walls was taken into account according to the recommendations in [6]. The friction was estimated based on the contact area of two vertical walls (thickness of perpendicular wall and half-length of a brick) and the pressure resulting from the weight transferred across this area.

Figure 3a) shows the activation coefficient of the mechanisms, while Figure 3b) shows the activation acceleration. For the case of wall Z1, it can be noted that it has the lowest values of activation acceleration in the general case without friction. This is the result of the generation of a greater horizontal force due to the greater weight of the wall, i.e. the forces that generate the seismic force. As the position of the line joint moves towards the higher floors, the value of activation acceleration increases, which is favorable. However, walls at higher floors are more prone to out-of-plane failure because they are subjected to higher values of acceleration, as higher floors contain different response frequencies and have different acceleration amplitudes than those at ground level. This is the result of a filtering effect in the dynamic response of the structure [7], and also higher walls are less pressed by the weight of above that stabilizes the mechanism. After taking into account the friction on the transverse walls, the considered wall Z1, it can be noted that its contribution is significant, especially up to a height of about 12 m. Beyond that, as the position of the line



joint moves upwards, due to the lower pressure force, but also due to the lower number of connections with the transverse walls the influence of friction decreases. For the gable wall itself, the effect of friction vanishes, which can also be seen from the overlap with the curve showing the activation values in the case without friction. The estimation of the kinematic behavior of wall Z2 is slightly more favorable than that of wall Z1 for the case without friction. Moreover, it can be noted that an additional condition for the formation of this mechanism is the failure at the failure of lintels, so that the kinematic chain can be activated.

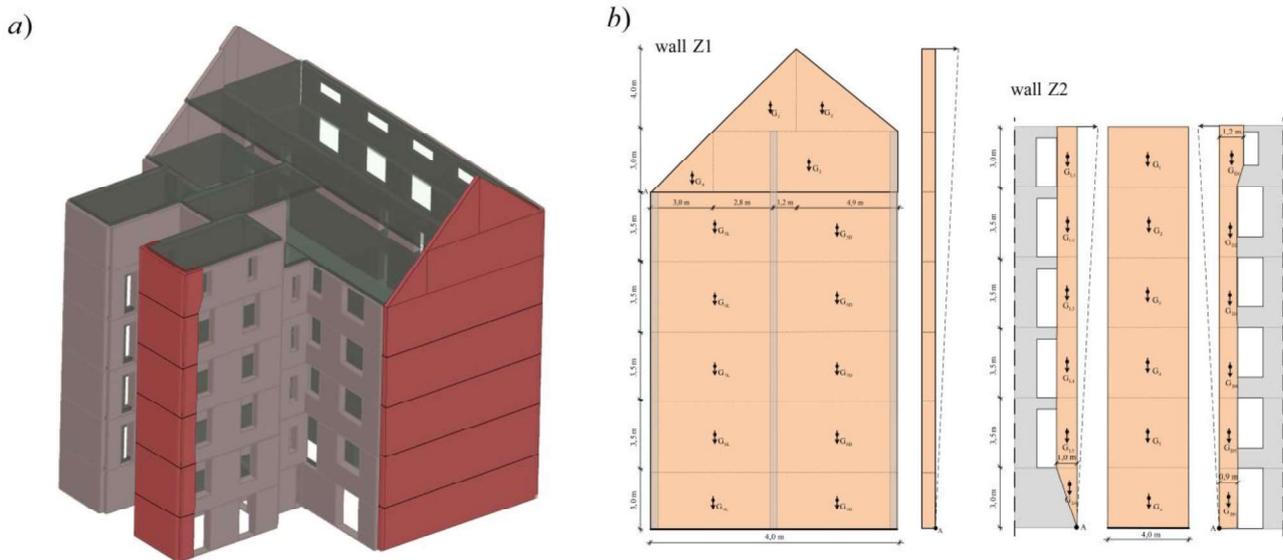


Fig. 2 – a) Building walls considered in the kinematic analysis b) mechanisms and the segments of the walls.

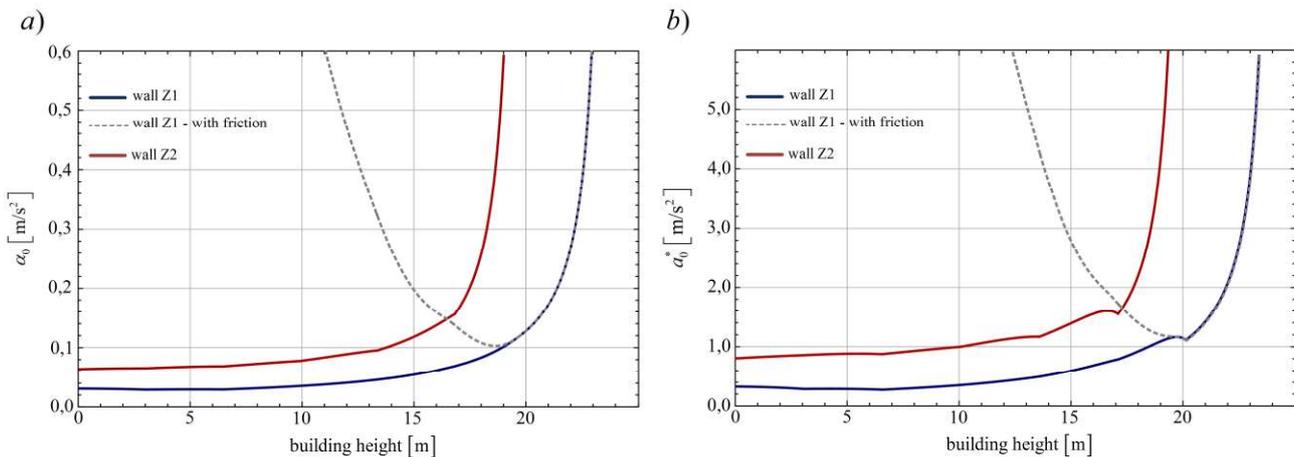


Fig. 3 – a) Activation factor diagram; b) activation acceleration diagram.

3.2 Response spectrum analysis

The modal analysis using the response spectrum was performed in accordance with the current standards for the earthquake resistance of new buildings (HRN EN 1998-1) and technical regulation for structures in Croatia for reconstruction after the earthquake. Design response spectrum for the return period of 95 and 475 years with the peak ground acceleration of 0.125g and 0.25g, respectively, are used in the analysis for limit state of significant damage (SD). All other assumptions were taken in accordance with HRN EN 1998-3. The



eccentricity of 5% was considered in the calculation, while the behavior factor was assumed to be 1.5 according to HRN EN 1998-3 for unreinforced masonry. The cracked sections of structural elements were taken into account because it is assumed that the building will reach a significant damage limit state. Due to high extent of results, only the most important ones are presented below.

The distribution of the storey shear force due to the action of an earthquake with a return period of 95 years for the X and Y directions is given in the Figure 4 a). Also in the same figure, the distribution of the shear forces in the case of an earthquake with a return period of 475 years, that is used for a new building design. For a return period of 95 years, the value of the base shear force in relation to the weight of the whole building is 15% for direction X and 16% for direction Y, while for a return period of 475 years it is 30% for direction X and 32% for direction Y. Average shear stresses per floor were also calculated and they amount to 0.14 and 0.16 MPa for the X and Y directions, respectively, for the 95-year return period. While the stress values were about twice as high for the 475-year return period. It should be noted that the shear stress calculated in this way can only be used for a rough estimate and does not represent a realistic distribution by structural element. It is important to emphasize that for the building under study, the ratio of storey shear forces and weight above the storey under consideration is not maximum in the basement and ground floor, but in higher storeys.

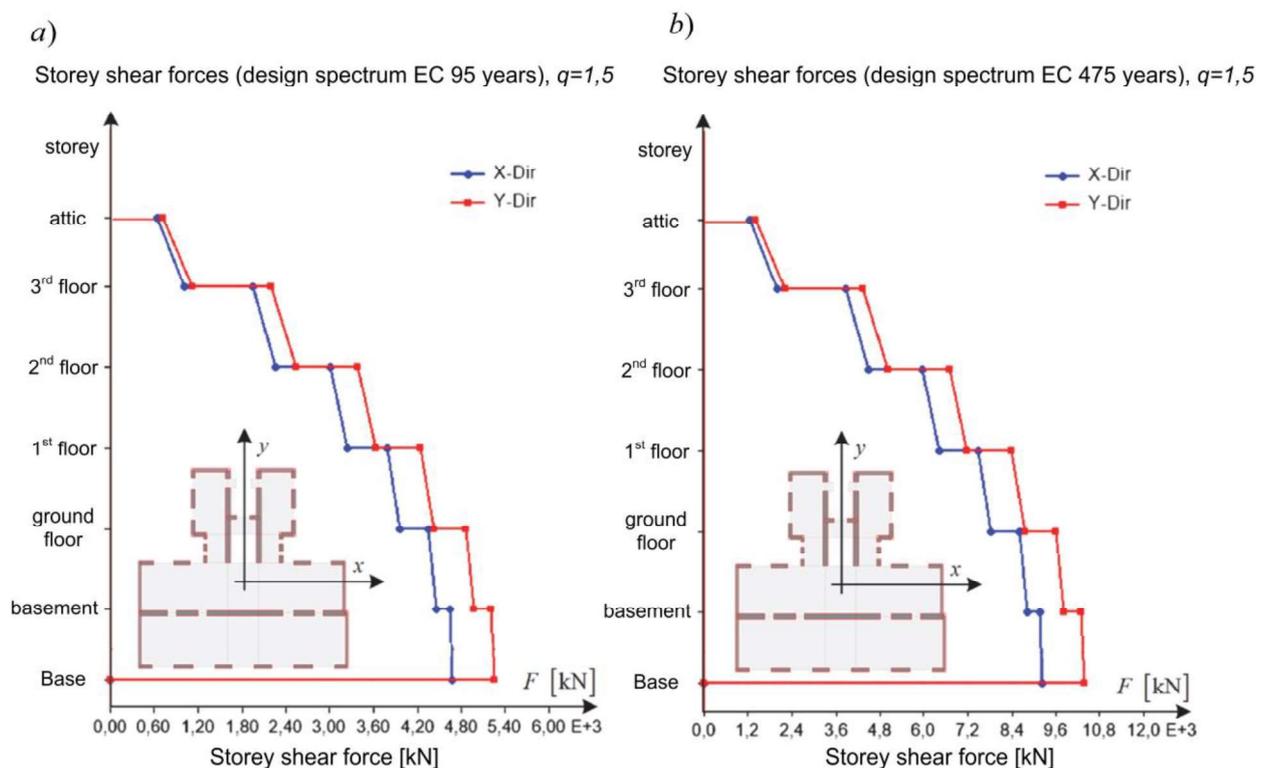


Fig. 4 – Storey shear forces for: a) 95 years return period design spectrum; b) 475 years return period design spectrum.

Since it is expected that the building cannot meet the conditions of the 475-year return period earthquake, the effect of the 95-year earthquake is analyzed in more detail. Namely, for this return period value ($a_g = 0.125$ g), a large number of elements seem to exceed the bearing capacity by more than 200%. It is worth highlighting that a part of the walls shows a load factor of more than 300%, which is well above the allowable values. Moreover, in order to exploit this type of linear analysis for internal forces, their redistribution to load-bearing elements from -25% to +33% was performed. The condition of such redistribution is that the resultant of the total forces on the storey remains the same (same intensity and position of the action). The results are presented in Figure 5, where it can be seen that the performed



redistribution has a rather favorable effect on the cross-sectional utilization. It should be emphasized that a 25% reduction in internal forces in the critical element does not necessarily mean a 25% decrease in utilization. Very often in unreinforced masonry, high forces greatly reduce the compressive area of the wall, which is crucial for the calculation of its resistance, so a 25% reduction can significantly increase the compressive area and thus significantly increase the resistance of the element. Another important thing to consider is the distribution of the 25% force from the critical element to another with the condition of the same resultant. If there are many walls in the plan and a high degree of static indeterminacy, this is usually not a problem. However, if there are a small number of walls in the floor plan, and in particular the presence of massive long walls that carry a significant load and are critical, the condition of equality of resultants is often not easily satisfied. However, even after redistribution, it can be seen that a significant portion of the walls exceed the bearing capacity even over 200%.

Response spectrum method ($a_g=0,125g$, $q=1,5 \rightarrow -25\%/+33\%$)

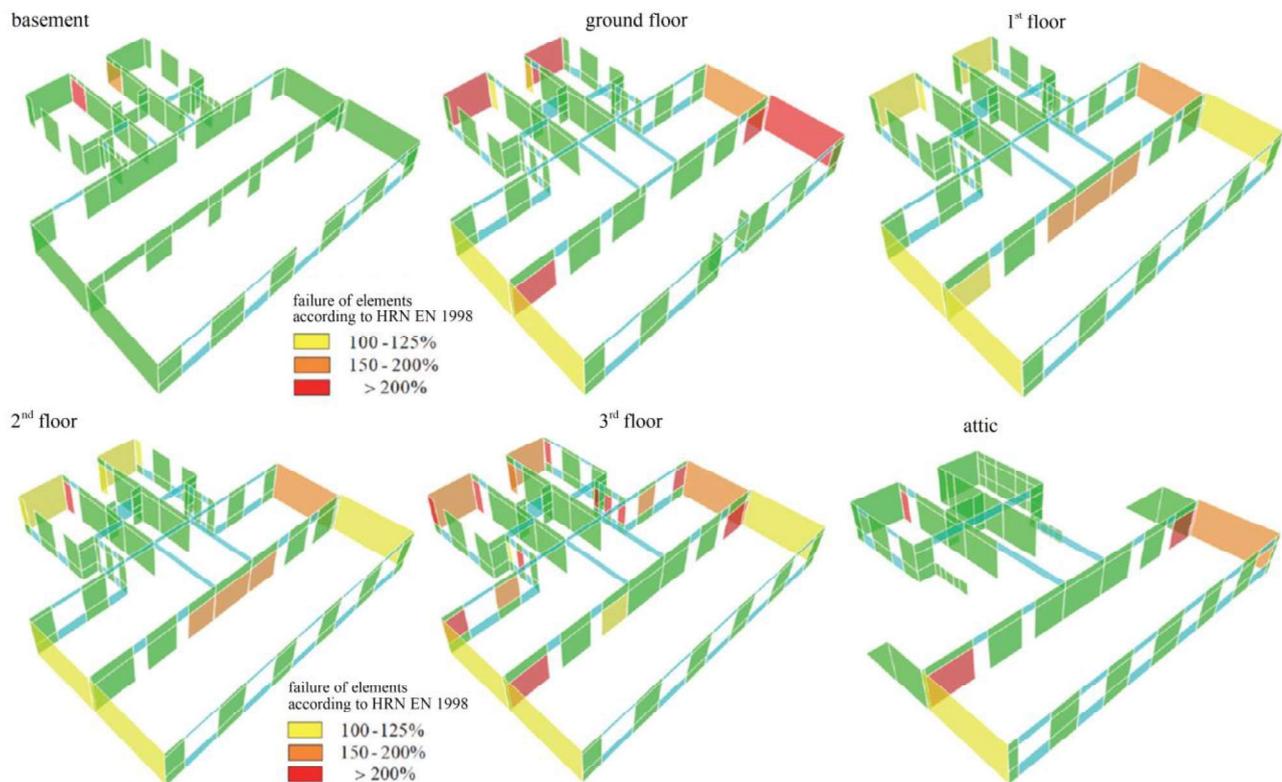


Fig. 5 – Utilization of wall cross-section with redistribution of transverse forces -25%% / + 33%

3.3 Pushover analysis

The numerical model created in the Etabs v.17 programme (Figure 6 a) was extended to define the nonlinear behavior of the masonry elements by defining capacity curves of the elements (walls and lintels), and a pushover analysis was performed by using the target displacement of the center of mass for different distributions along the height of the structure. The forms of the acceleration distribution over the building are shown in Figure 6 b). The direction of the load is applied in both X and Y directions considering the positive and negative sign of the load. Furthermore, the eccentricity value of $\pm 5\%$ of the perpendicular plan dimension to the center of mass is considered.

Figures 7 and 8 show all the representative pushover capacity curves obtained. A single family of curves belonging to a particular lateral load distribution is marked. From the figures it can be deduced which load patterns are most unfavorable for the building behavior. It can be seen that the same load distribution shapes



are relevant for both load directions (with a slightly smaller or larger difference). The most unfavorable load distribution is in the form of a cantilever deformation line (KONZ) and is represented by a dashed line. This form of load is extremely unfavorable and very unlikely and was not considered relevant. Therefore, a linear load distribution versus height (LIN) can be assumed to be most informative. It corresponds approximately to the distribution of horizontal forces according to the lateral force analysis method. The structure behaves similarly, but somewhat more favorably, with the load distribution according to the deformation line of the shear building, which is a parabola (POSM). The most favorable response and the maximum bearing capacity of the structure was obtained for the lateral load proportional to the inertial forces caused by the constant acceleration of the masses. This form, along with the first vibration mode in this direction, is mandatory for checking the calculation when performing the pushover analysis. The shape proportional to the oscillation shape that has significant mass participation in a given direction (1st mode_X, 1st mode_Y) proved to be almost equal to the linear distribution.

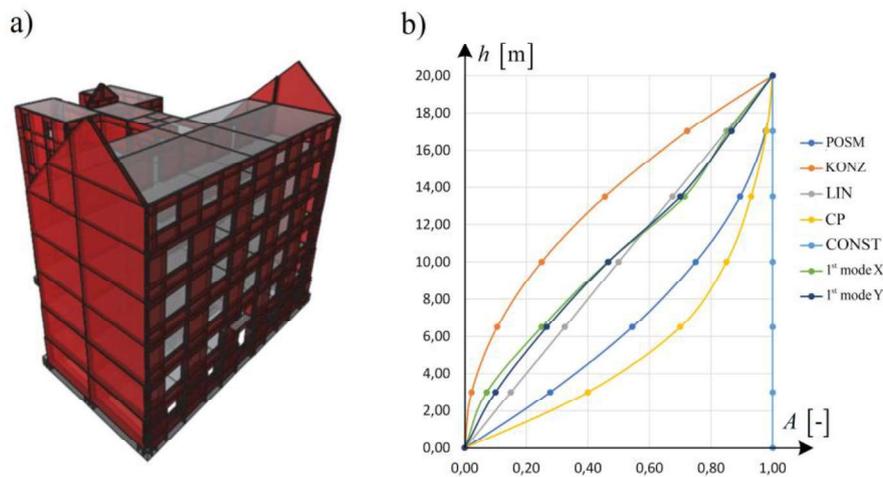


Fig. 6 – a) Numerical model in Etabs v.17; b) lateral force patterns

In the first stage of pushover analysis for the positive direction of the LIN form of the load distribution in the X direction, several structural elements reach a state of limited damage up to a force of approximate 2000 kN, which corresponds to the base shear factor of 6,2%. The first failure of structural elements occurs at a lateral force of 3200 kN and several lintels reach the state of significant damage. The first wall failure occurs at the 3rd floor at a force value of 4000 kN (B.S. = 12.5 %) and the CM displacement of the 3rd floor of 5.1 cm. The bearing capacity of the walls in the higher floors is reduced compared to the lower floors, which is a consequence of the smaller value of the axial force. The next important point is the exceeding of the significant damage limit state due to the failure of the walls in the lower floors. The global failure occurs at a lateral force value of 4200 kN (B.S. = 13%) and a CM of the 3rd floor 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 fail, reaching the significant damage limit state. The capacity curves in the negative X direction of lateral loading show a similar trend of the occurrence of lintels damage. In this case, the failure mechanism of the structural system starts with a failure of the western courtyard wall at the 1st and 2nd floors by reaching a significant damage limit state for the value of the lateral force of approximately 3900 kN.

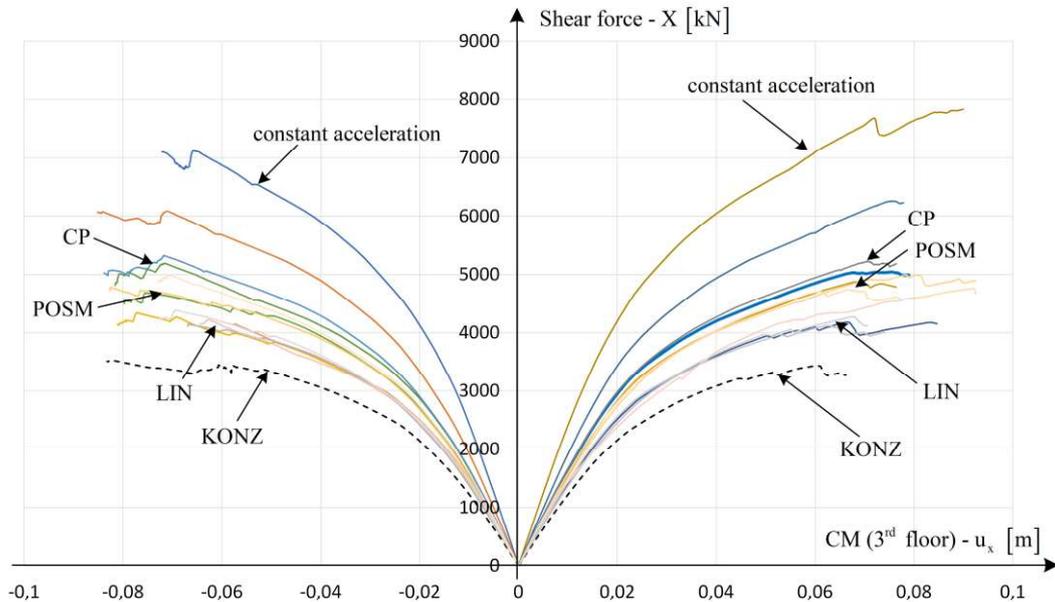


Fig. 7 – Pushover curves for direction X

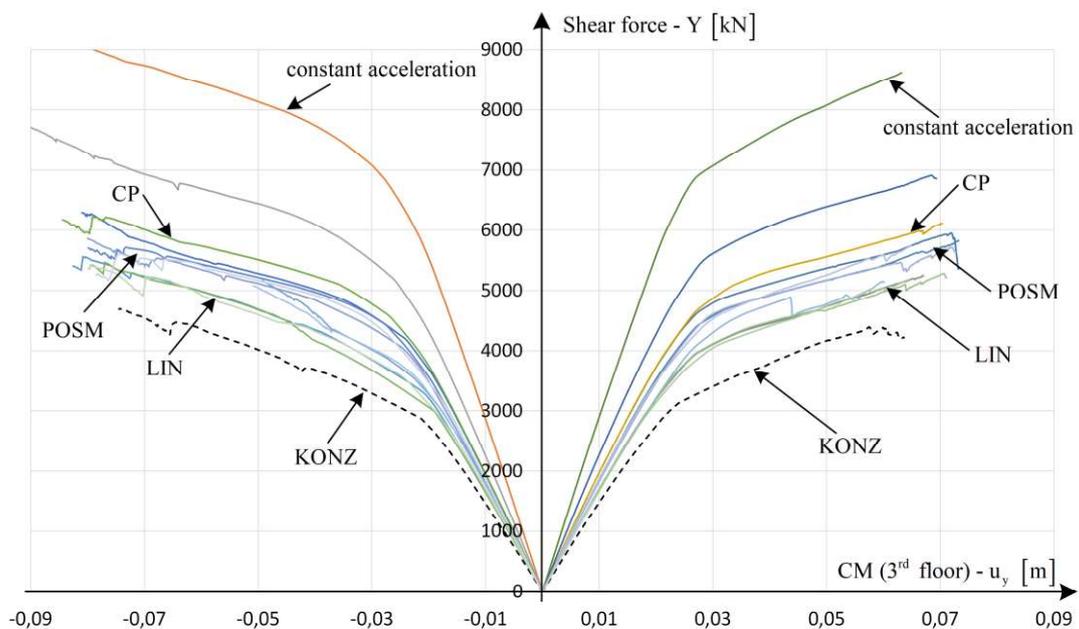


Fig. 8 – Pushover curves for direction Y

For the load distribution in the Y direction, if we neglect the form KONZ, the applicable load distribution for the direction with positive and negative sign is the LIN. The condition of limited damage for several structural elements is reached for about 2600 kN lateral force (B.S. = 8.1%) and displacement of the 3rd floor CM of 1.6 cm. The first damaged elements are located on the 3rd floor and the damage progresses with increasing load, especially in the lintels. The onset of structural element failure occurs at a force of 4500 kN (B.S. = 14%) and a displacement of the 3rd floor CM of 4.3 cm. The critical elements that first reach a significant damage limit state are the gable walls on the north side of the building. These are the 1st floor walls, followed by the exceeding of the bearing capacity of the 2nd floor gable walls. Since the symmetry of the building holds in this direction, the south gable walls can also be critical elements, depending on the direction of load eccentricity. On the curves, a sudden drop in the bearing capacity curve can be observed,



caused by the failure of a part of the gable wall. Further damage continues for the courtyard façade walls at the 2nd and 3rd floors. As in direction X, we have the critical wall failures at the higher floors of the building. The ultimate displacement of the 3rd floor CM in the global building collapse occurs is approximately 6.0 cm.

Bearing capacity and building deformation requirement by reducing the system to an equivalent system with one degree of freedom were also considered for the building under consideration. The procedure is carried out according to the N2 method [8, 9]. A requirement for a peak ground acceleration of 0.125 g on a bedrock for limit state of significant damage (SD) which corresponds to a return period of 95 years is adopted in accordance with the technical regulation for structures in Croatia for reconstruction after the earthquake. For each of the 72 load cases, a bearing capacity curve was obtained and the verification for the peak ground acceleration was performed. Relevant curves and important parameters are shown in Figure 9. Results are shown for the X and Y directions for the case of a linear lateral force distribution (LIN). Important parameters are marked and it can be observed that the deformation capacity of the system for both directions is higher than the required capacity and that some load capacity reserves are present for the case without eccentricity.

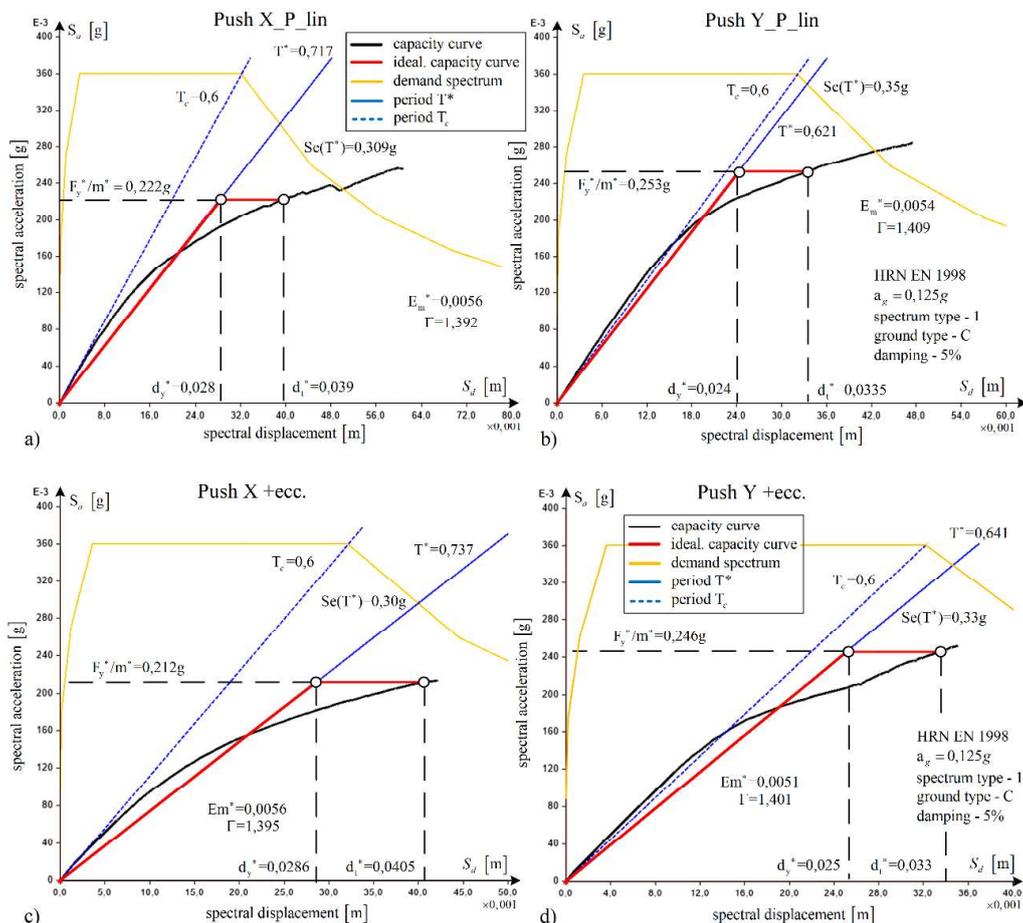


Fig. 9 – Capacity curve and displacement requirement of an equivalent 1DOF system

If eccentricity is taken into account, the capacity of the building decreases significantly. Diagram (c) shows an idealized capacity curve of the 1DOF system for a peak ground acceleration of 0.125g considering the X-direction of the lateral force distribution (X_P) in the presence of an eccentricity of 5% of the building plan dimension. The deformation capacity of the system is approximately equal to that required and for this case the system has no additional load carrying capacity. The maximum ground acceleration that satisfies the deformation requirement is approximately 0.125 g, and this load case has been found to be most relevant.



Diagram d) shows an idealized load capacity curve of an equivalent 1DOF system with a ground acceleration of 0.125 g and Y-direction of lateral load distribution according to the lateral force method (Y_P) in the presence of an eccentricity of 5% of the building length. Again, it can be concluded that the deformation capacity of the system is approximately as required, and the system has no additional load carrying capacity.

4. Conclusion

The building seismic performance is evaluated using methods of different complexities that include a local kinematic analysis of gable walls, a linear response spectrum method commonly used in engineering practice, and a nonlinear pushover analysis. The kinematic analysis was performed parametrically considering different positions of line joint formation. Based on the parametric calculations, it is expected that the position of the line joint at the very bottom of the structure gives the smallest values of activation acceleration. However, we emphasize that for wall Z1 there is a friction effect, which is more pronounced at the position of the line joint below half building height. Further, for wall Z2 a complete failure at weak points, or cracking of all lintels is assumed. With increasing height, the values of acceleration increase, but the possibility of triggering the mechanism in this case depends significantly on the response of the whole structure, since at higher floors the response is affected by the natural frequencies of the building, especially dominant modes. It is important to emphasize that the obtained results include only the activation acceleration values based on the linear analysis and not the displacement capacity after the activation of the mechanism. By performing calculations based on response spectrum analysis, commonly used for new structures, a value of lateral force of 15% of the weight for direction X and 16% of the weight of the building for direction Y was obtained for a return period of 95 years (with importance factor 1.0 and behavior factor 1.5). Such a level of shear forces causes significant local overloads. Although internal force redistribution is expected, it cannot be satisfactorily accounted for by the response spectrum calculation. Moreover, redistribution of internal forces has shown that the most critical elements are the gable walls, which highly exceed the utilization of the cross sections. Nonlinear calculations using the pushover method resulted in more detailed analyzes and estimates of the load carrying capacity of the building. The linear load distribution was found to be relevant. It should be noted that the limit state analysis is performed at the level of the whole structure and at the level of the element. If an element fails before the global load capacity reserves are exhausted, that element should be strengthened, and adequate deformation capacity should be ensured. It is important to note that in case of failure of a particular element, any part of the floor structure must be secured against local collapse to prevent risk to human life.

Based on the presented results of a detailed numerical analysis, an engineering estimate of the expected behavior of the building due to different earthquake scenarios can be given. Importantly, this estimate also applies to the case where only minimal measures have been taken to satisfy the numerical model assumptions. In addition to the usual assumptions of rigid diaphragms preventing in out-of-plane failure of walls, it must be assumed here that damaged walls have been reconstructed and their load-bearing capacity restored at the time before the earthquake damage:

- 95 year return period earthquakes may cause cracking or localized damage to the previously mentioned critical structural elements, but no significant damage to structure walls or local failures are expected. According to the definitions of the boundary conditions, it can be said that a large portion of the load-bearing elements will be in a state of limited damage.
- 475 year return period earthquakes impose demands that far exceed the capacity of the structure and can be expected to cause significant load-bearing capacity exceedances, local and global collapses, and loss of building stability.

At present, the condition of the building is such that it has been damaged by the earthquake. The cause of damage is mainly the lack of rigid or at least partially rigid diaphragms in the floors that are well connected to the surrounding walls. This led to the occurrence of local out-of-plane wall failure mechanisms of the parapet roof wall. Aside from the local failure mechanisms, the most severe damage is to the west walls in the courtyard portion of the building where significant diagonal cracking has occurred. Apart from the fact



that these walls are at the edge and have the highest displacement demand, which they cannot reach, they are not well connected to the floor slabs. Although there is a thin concrete slab at these locations, the underlying beam is parallel to the wall and not connected to it. An additional disadvantage is that it is a practically independent wall because there are very large openings and weakenings on the vertical walls that connect to it at the corners. Although it is a relatively long wall with no openings, the flow of forces in it is not regular, which is concluded on the basis of the diagonal cracks that occurred. The truss analogy is not realized because the diagonal compressive force transmitted by the wall at floor level cannot be balanced with the horizontal tensile force to be transmitted to the beginning of the wall. This exceeds the tensile strength of the masonry and opens cracks.

Seismic retrofitting measures should improve the seismic performance of the building, which is shown by the calculation results, and it involves strengthening the horizontal structures to act as rigid or semi-rigid diaphragms. This is one of the most important measures, as the diaphragms ensure a uniform distribution of the seismic load on the walls and prevent the formation of local out-of-plane failures of the walls. It is particularly important to provide a connection with the gable walls. Repair and reinforcement of all load-bearing elements damaged by the earthquake, as well as all walls that proved to be critical in the calculations performed. The reinforcement of the lintel is recommended to prevent its complete disintegration, which could endanger human lives. The purpose of the intervention in the lintel is to ensure compactness during and after the earthquake. In addition to the reinforcement of the primary seismic elements, it is important to consider the secondary elements (non-load-bearing walls), which must be properly connected to the load-bearing structure in order to ensure its local stability.

5. Acknowledgements

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6. References

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