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Case-study of the out-of-plane wall failure of a typical downtown building in Zagreb

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Abstract

Out-of-plane wall failures was recognized as the most common form of earthquake damage to buildings that occurred during ML 5.5 earthquake that hit the City of Zagreb on Sunday, 22nd March 2020. In this paper, a typical residential building in city centre known as Lower Town is studied. Post-earthquake building survey showed that one gable wall partially failed, while damage on several other walls showed a tendency to separation with the possibility of overturning. In the numerical analysis, a linear and nonlinear procedure that are based on rigid body model and the method of virtual work is used. Requirements for spectral acceleration and spectral displacement are based on recommendations given in Italian regulation NTC2008. Peak ground acceleration recorded in Zagreb earthquake is used for the verification of the results.

Key words: unreinforced masonry, local mechanisms, linear model, nonlinear model

1. Introduction

The earthquake that hit Zagreb, the capital city of Croatia, on March 22, 2020 at 6:24 local time was magnitude 5.5, according to Richter. The epicenter was located 7 km north of the Zagreb historic centre. The earthquake, although of moderate magnitude, caused great material damage, especially to the buildings in the historic core of the city. The building fund of the City of Zagreb includes a large number of traditional unreinforced masonry buildings. This kind of buildings are most often consisting of interconnected load-bearing masonry walls and flexible horizontal structures with wooden beam elements. The damage to this type of structures during earthquakes most often occurs due to uneven distribution of seismic forces due to flexible floor structures, inappropriate or non-existent interconnections of elements, and poor contact with the roof and floor structure [1]. Postearthquake survey of buildings in Zagreb showed that the most common damage to this kind of masonry buildings in the city center is associated with local damage to the roof (Figure 1.b.), collapse of attic chimneys (Figure 1.a), and partial or complete collapse of gable attic walls (Figure 1.c) [2].

Figure 1. a) Collapse of chimneys; b) roof damage; c) collapse of attic gable wall [2]

In addition to the damage of the gable walls and roof, the damage to the walls due to exceedance of in-plane bearing capacity in its own plane is also observed. It manifested as diagonal cracks in the partitional and load-bearing walls, and as cracks of the lintels. The cracks were also evidenced along the connections of perpendicular walls that can seriously endanger structural integrity by making walls susceptible to the out-of-plane failure.

One of the main observed seismic vulnerability of buildings in the Zagreb center is the appearance of local mechanisms, known as out-of-plane wall failure. For such mechanisms are characterized with vertical cracks that appear at the wall connections and the horizontal cracks at the connection of the floor structure and the wall [2]. If a rocking of the wall occurs, it poses a great danger to the structural stability and human lives. It is one of the reasons that make the out-of-plane wall failure one of the key aspects in assessing the seismic vulnerability of unreinforced masonry structures. The purpose of this paper is to provide a critical overview of methods used for determining the conditions of out-of-plane wall failure applied to existing masonry downtown building in Zagreb.

During the recent years, extensive research is ongoing regarding out-of-plane wall failures [3,4]. Some countries, such as Italy, have gone a step further by including verification procedures for this kind of mechanisms in technical guidelines [5]. Although Eurocode 1998-3 guidelines currently does not prescribe these checks, it is known that the new version will contain procedures for out-of-plane wall failure.
In this paper, a case-study of out-of-plane wall failures of a typical unreinforced masonry building is presented. For several walls, the out-of-plane mechanisms were recognized and analysed using linear and nonlinear model.

2. Analysis procedure

Separation of the walls from the rest of the structure due to cracks cause the formation of a mechanism that is susceptible to motion due to inertial action. In the analysis, for these walls is assumed that a kinematic chain if formed composed of rigid or semi-rigid blocks (Figure 3.a). The selection of the type of mechanism is based on engineering assessment and experience gained in previous earthquakes, inspection and analysis of cracks, and the typology of buildings in the area [6]. For example, connections with the vertical (perpendicular walls) and horizontal (floors) elements, material and type of connections between elements, weakening of elements such as smaller wall thickness at parapets, anchoring of elements, tension and friction, arch and roof induced horizontal forces and other specific influences. Direct observation of crack patterns to the buildings due to earthquakes helped to identify several characteristic mechanisms, which are shown in Figure 2 [2].

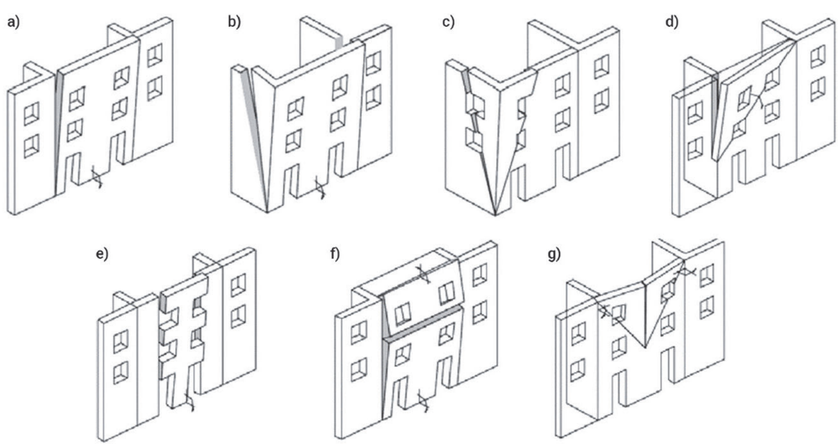


Figure 2. Out-of-plane failure patterns [2]

Kinematic chain of blocks is defined as the one-degree-of-freedom system (Figure 3.a). After determining the geometry of the kinematic chain and boundary conditions, it is important to take into account all the forces that affect it, including the inertial force induced by the acceleration of the block. The activation of the mechanism is determined by employing the equation of virtual work:

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

In Equation (1), P_i are the weights of the blocks involved in mechanism, P_j are the forces that are transmitted to the certain block, F_h is the general horizontal force that act on the block (usually induced by the thrust of vaults), $\delta_{x,i}$ is the horizontal virtual displacement, and $\delta_{y,i}$ vertical virtual displacement. L_{fi} is the virtual work of the internal forces (eq. friction, tenson forces). The coefficient α_0 multiplies all forces that are induced by block interion.
In order to determine the acceleration that activates the mechanism, the effective mass M^* and effective mass ratio e^* of the 1DOF system needs to be determined by the following expression:

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

Finally, spectral acceleration of 1DOF system that activates mechanism in motion is determined from the expression:

$$a_0^* = \frac{\alpha_0 g}{e^*\,\mathrm{F_C}} \tag{3}$$

In the expression (3) the $\mathrm{F_C}$ represents the confidence factor. When using assumption of the rigid blocks the NTC2008 guidelines [5] propose the value of 1,35. The value of spectral acceleration a^*_0 is used for the verification of mechanism activation according to the linear procedure. The model that assumes absolutely rigid blocks is bi-linear (Figure 3.b). The initial stiffness is infinite until the maximum value of the lateral force is reached. This value of force activates the mechanism in motion. In the limit position in which the system is unstable the value of the force vanishes. However, several investigations showed that this kind of verification proved to be conservative since the activated blocks have a certain displacement capacity before overturning (Figure 3.b). Research done by Doherty [4] confirmed that individual masonry blocks can deform significantly when exposed to high load values from the upper storeys. Therefore, the semi-rigid block model has tri-linear diagram and based on this behaviour, the basic idea of a nonlinear model follows. In the region of the plateau the system can still be considered stable.
To take into account nonlinear behaviour of the mechanism the control displacement d_k is introduced and equivalent spectral displacement d^* is determined by the following transformation:

$$d^* = d_k \frac{\sum_{i=1}^{n+m} P_i \, \delta_{Px,i}^2}{\delta_{x,k} \sum_{i=1}^{n+m} P_i \, \delta_{Px,i}} \tag{4}$$

In the simplified case, when all actions (weight, external or internal forces) remain almost constant as the kinematics of the mechanism develop, the curve that describes relation of the activation factor and control displacement is almost linear, so it can be defined by $\alpha = \alpha_0(1 - d_k / d_{k,0})$. $d_{k,0}$ is the maximum displacement that corresponds to the vanishing value of activation coefficient α. Therefore, the value of the spectral displacement that corresponds to the spectral acceleration of equivalent system can also be determined using linear relation:

$$a^* = a_0^* \left(1 - \frac{d^*}{d_0^*} \right) \tag{5}$$

Finally, the displacement capacity demand prescribe the value of the ultimate spectral displacement as well as value of spectral displacement and spectral acceleration for the determination of secant period of the equivalent system. In the Italian NTC 2008 guidelines [5] the following values are used: for ultimate displacement $d^*_u = 0.4\, d^*_0$, where d^*_0 is the limit displacement of equivalent system that is related to the limit displacement of the kinematic chain d_0, and the value of the displacement $d^*_s = 0.4\, d^*_u$.

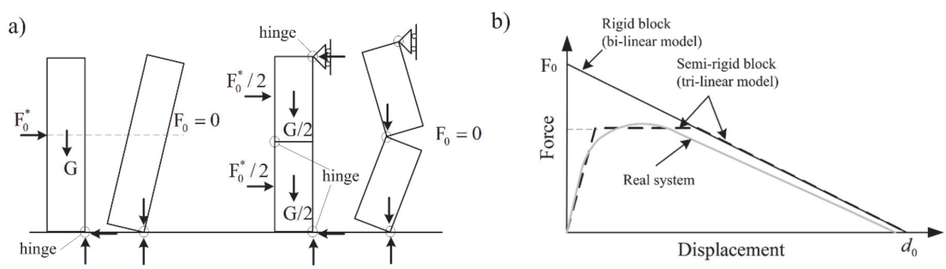


Figure 3. a) Rigid blocks mechanisms; b) Force-displacement relation for the mechanisms [4]

To conduct the verification checks, it should be noted that accelerations on higher floors contain different frequencies and have different acceleration amplitudes than those at ground level. This occurs due to the building dynamic response which has a filtering effect on the accelerations of individual floors [7]. Therefore, the accelerations that cause the walls to overturn depend on the position of the wall within the structure. Usually, the acceleration amplitudes increase with the height of the structure and while the excitation on the ground floor of the building has a very diverse frequency composition, the response on higher floors is usually influenced by the first mode of oscillation. According to the [5], the spectral acceleration of equivalent system needs to satisfy following expression:

$$a_0^* \geq a_{0,\min}^* = \max\left(a_g\, S / q; S_e(T_1)\psi(z)\Gamma_1 / q\right) \tag{6}$$

When nonlinear procedure is used, the spectral displacement capacity of the mechanism according to [5] is estimated from the expression:

$$d_u^* \geq d_{u,\min}^* = \max\left(S_{De}(T_s); S_{De}(T_1)\psi(z)\Gamma_1 \left(\frac{T_s}{T_1}\right)^2 / \sqrt{\left(1-\frac{T_s}{T_1}\right)^2 + 0{,}02\frac{T_s}{T_1}} \right) \quad (7$$

3 Case study: Buidling in Mrazoviceva street

The case-study unreinforced masonry building is built 1911 and it is located in the center of Zagreb. The building is built in block (Figure 4.a). The building has a basement, 4 storeys and the attic. The plan dimensions are 18,20 x 11,35 m (Figure 4.b) and the building height is 17,5 m. The structural system consists of interconnected solid brick walls and floor system are timber joists except for the basement where it consists of concrete slab with steel girders. The load-bearing walls are made of solid brick of the old format (290 × 140 × 65 mm) that are 75, 60, 45 and 30 cm thick. The thickness of the walls narrows with the height of the building. The layout of the walls and the structural system are classic for this period of construction. The post-earthquake survey showed that cracks are mostly present on the non-structural elements, parapets, partition walls, and lintels. Severe damage suffered chimneys and the west gable wall that partially collapsed (Figure 5.a). During inspection, the cracks were observed along the connection of façade wall with floors and perpendicular wall which indicated out-of-plane displacement of the façade wall (Figure 5.c).

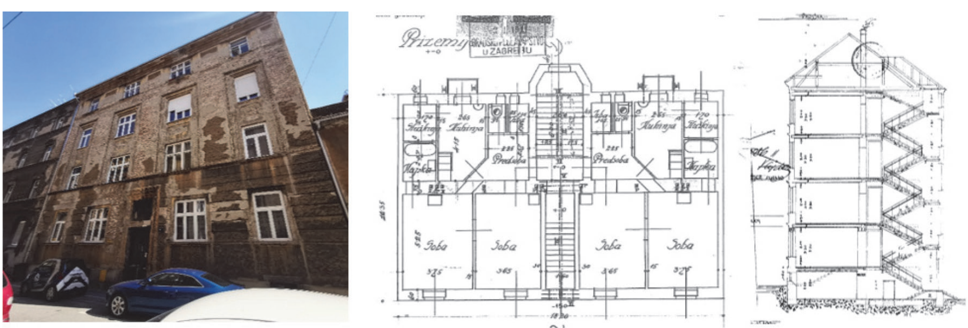


Figure 4. a) Case-study building in Mrazoviceva Street; b) Archive architectural drawings

3.1 Out-of-plane vulnerability assessment

Figure 6.a) shows a three-dimensional view of selected walls that are analysed. All walls were modelled as single rigid block since there is no strengthening elements in the levels of floors that could lead two body motion. For all mechanisms, the plan of virtual displacement is determined for the unit value of control node displacement. This point is also used as control point in the nonlinear analysis. Detail numerical model of the build-

ing can be found in the reference [8], and here we use only the following parameters: volume weight of masonry γ = 18 kN/m^3, first period value T_1 = 0,603 s, and modal participation factor Γ_1 = 1,364. The following walls are considered: mechanism 1 – attic gable wall; mechanism 2 – gable wall of 3rd floor including attic; mechanism 3 – façade wall on street side; mechanism 4 – façade wall on courtyard side. The verification is based on the NTC2008 guidelines [5] for which we used recommended parameters: structural coefficient q=2 and confidence factor F_C=1,35. Peak ground acceleration used in the analysis is a_g=0,22g which was the maximum acceleration that was recorded during the Zagreb earthquake in March 2020 [1].

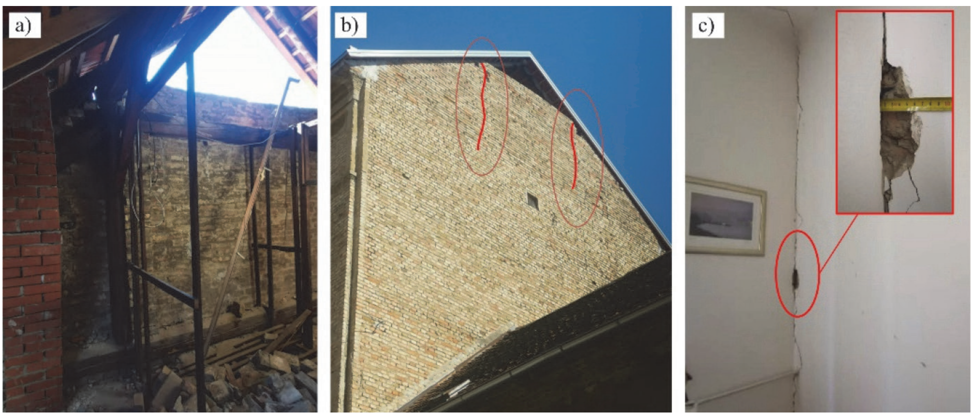


Figure 5. a) Gable wall inside the attic; b) Crack patterns of gable wall; c) Cracks on the connection of facade wall and perpendicular wall

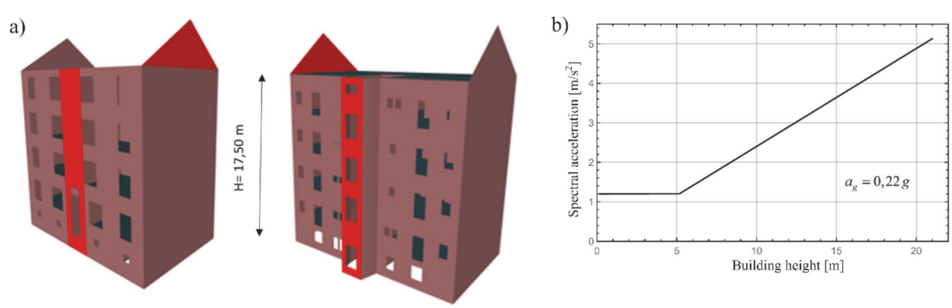
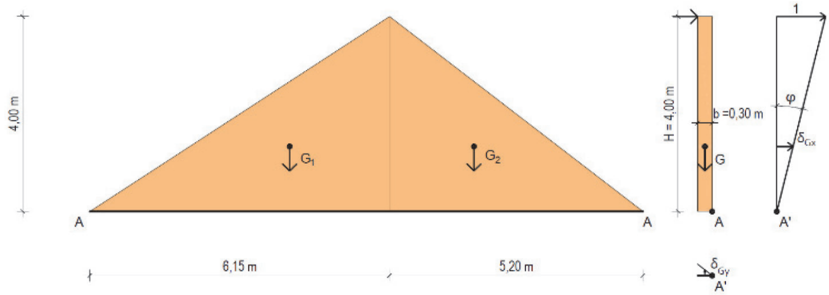


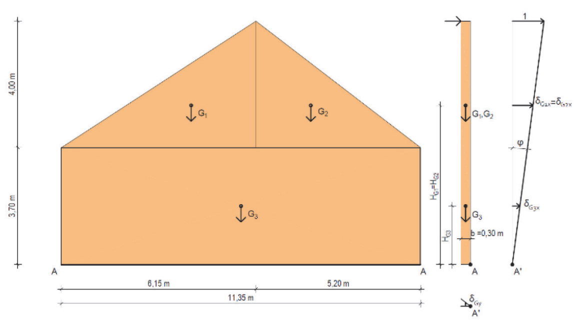
Figure 6. a) 3D view of the analyzed mechanism; b) Acceleration level requirement

Table 1. Mechanism 1 – attic gable wall



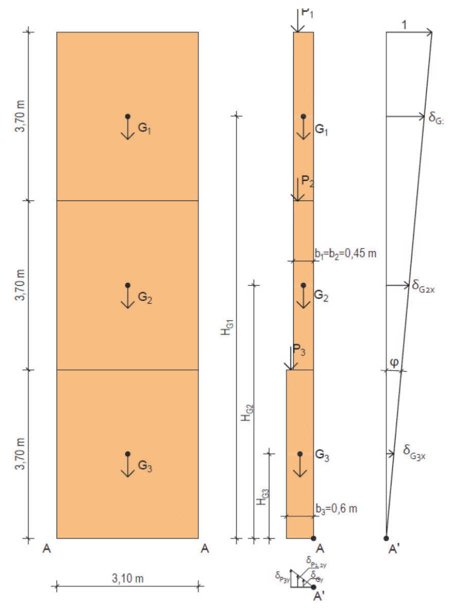
Model parameters	Linear model parameters	Nonlinear model parameters
G_1 = 66,42 kN; G_2=56,16 kN $\delta_{G1,x}$= $\delta_{G2,x}$ = 0,333, $\delta_{G1,y}$=$\delta_{G2,y}$ = 0,037 z=17,50 m	α_0=0,11; a^*_0=0,81 m/s^2 $a^*_{0,min}$ = 4,75 m/s^2 a^*_0/ $a^*_{0,min}$ =0,17	$d_{k,0}$ = 0,45 m; d^*_0 = 0,15 m d^*_u = 0,66 m d^*_s = 0,024 m T_s=1,18 s $d^*_{u,min}$ = 0,3 m d^*_u/ $d^*_{u,min}$ = 0,2

Table 2. Mechanism 2 – gable wall of 3rd floor including attic



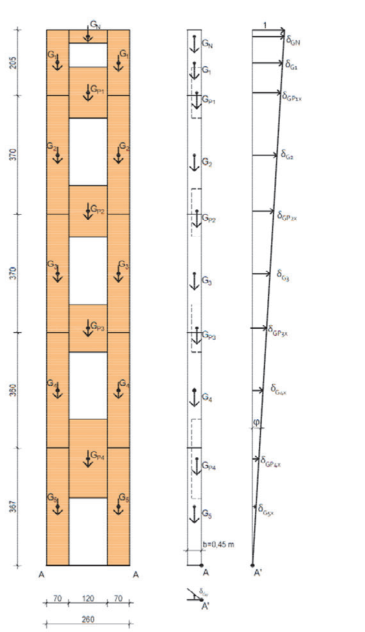
Model parameters	Linear model parameters	Nonlinear model parameters
G_1 = 66,42 kN; G_2=56,16 kN; G_3=214,98 kN $\delta_{G1,x}$= $\delta_{G2,x}$ = 0,653, $\delta_{G1,y}$=$\delta_{G2,y}$ = 0,019 $\delta_{G3,x}$= 0,240, $\delta_{G3,y}$ = 0,0,019 z=13,70 m	α_0=0,049; a^*_0=0,451 m/s^2 $a^*_{0,min}$ = 3,74 m/s^2 a^*_0/ $a^*_{0,min}$ =0,12	$d_{k,0}$ = 0,385 m; d^*_0 = 0,19 m d^*_u = 0,08 m d^*_s = 0,03 m T_s=1,77 s $d^*_{u,min}$ = 0,17 m d^*_u/ $d^*_{u,min}$ = 0,47

Table 3. Mechanism 3 – façade wall on street side



Model parameters
$G_1 = G_2 = 92{,}91$ kN; $G_3 = 123{,}87$ kN; P = 16, 93 kN;
$\delta_{G1,x} = 0{,}83$; $\delta_{G2,x} = 0{,}5$; $\delta_{G3,x} = 0{,}167$;
$\delta_{P1,x} = 1{,}0$; $\delta_{P2,x} = 0{,}666$; $\delta_{P3,x} = 0{,}333$;
$\delta_{G1,y} = 0{,}02$; $\delta_{G2,y} = 0{,}02$; $\delta_{G3,y} = 0{,}027$;
$\delta_{P1,y} = 0{,}03$; $\delta_{P2,y} = 0{,}03$; $\delta_{P3,y} = 0{,}045$;
z=6,40 m.
Linear model parameters
$\alpha_0=0{,}049$; $a^*_0=0{,}47$ m/s^2
$a^*_{0,min} = 1{,}56$ m/s^2
$a^*_0 / a^*_{0,min} = 0{,}30$
Nonlinear model parameters
$d_{k,0} = 0{,}556$ m; $d^*_0 = 0{,}366$ m
$d^*_u = 0{,}146$ m $d^*_s = 0{,}06$ m
$T_s=2{,}43$ s
$d^*_{u,min} = 0{,}149$m
$d^*_u / d^*_{u,min} = 0{,}98$

Table 4. Mechanism 4 – façade wall on courtyard side



Model parameters
$G_1 = 11{,}66$ kN; $G_2 = 20{,}90$ kN; $G_3 = 20{,}41$ kN; $G_4 = 20{,}41$ kN;
$G_5 = 20{,}82$ kN; $G_N = 3{,}89$ kN; $G_{P1} = 10{,}37$ kN; $G_{P2} = 9{,}72$ kN;
$G_{P3} = 9{,}72$ kN; $G_{P4} = 15{,}98$ kN;
$\delta_{G1,x} = 0{,}942$; $\delta_{G2,x} = 0{,}769$; $\delta_{G3,x} = 0{,}547$; $\delta_{G4,x} = 0{,}328$;
$\delta_{G5,x} = 0{,}110$; $\delta_{GN,x} = 0{,}991$; $\delta_{P1,x} = 0{,}880$, $\delta_{P2,x} = 0{,}658$;
$\delta_{P3,x} = 0{,}436$; $\delta_{P4,x} = 0{,}220$;
$\delta_{P1,y} = \delta_{P2,y} = \delta_{P3,y} = \delta_{P4,y} = 0{,}009$;
z = 0 m
Linear model parameters
$\alpha_0 = 0{,}107$; $a^*_0 = 0{,}47$ m/s^2
$a^*_{0,min} = 1{,}24$ m/s^2
$a^*_0 / a^*_{0,min} = 1{,}01$

For mechanism 1, only 17 % of the linear model demand and 20 % of the nonlinear model demand is fulfilled. This wall, however, only partially collapsed in the earthquake in March 2020. The model used in the analysis assumes complete separation of the wall from the rest of the structure, but it can be seen from the photographs on Figure 5. a) and b) that in the thirds of the length the wall was thickened with an additional layer of bricks, that certainly helped to prevent complete separation. It should be noted that cracks exist in these parts of wall that are visible from the outside (Figure 5.b). We also emphasize here that the main direction of the earthquake that happened, was approximately parallel to the gable wall plane, which certainly influenced the maximum value of the acceleration that affected this wall. The same wall was analysed in mechanism 2 were it includes the part of the wall on the storey below the attic. In the linear model, there is the fulfilment of 12 % of the demand, while the displacement capacity demand is fulfilled with 47 %. For this wall the connections with perpendicular walls is not completely impaired, so there are some reserves in terms of friction force that is present at the junction of the walls that was not considered in the analysis. The results of street façade wall considered in mechanism 3 showed that the wall should have been most likely activated during earthquake since linear model had fulfilment of 30 %. However, in the nonlinear model, the model had almost complete fulfilment of the demand that was 98 %. This wall, unlike the gable wall, was exposed to a greater acceleration values when considering the main direction of the earthquake. The façade wall on the courtyard side, mechanism 4, have satisfied requirement of the linear model so it was concluded that it was not activated in earthquake, therefore, the nonlinear model was not considered.

4 Discussion

This paper showed a case-study of out-of-plane seismic behaviour of unreinforced masonry walls by different methods of local analysis. The linear analysis proved to be more conservative than the nonlinear analysis. It was showed that all considered walls except the façade wall on the courtyard side can be activated due to peak ground acceleration of 0,22*g*. The gable wall that partially collapsed in the earthquake is the most vulnerable to out-of-plane failure that was confirmed by linear and nonlinear procedure.

Out-of-plane wall failure is the primary analysis when considering unreinforced masonry buildings, especially when it comes to the buildings that are damaged in an earthquake. If the verifications done by linear or nonlinear procedure is not fulfilled the results of the global structural response become unreliable. Although most of technical guidelines, including Croatian regulations, does not require these verifications, so the engineers must take into account the structural retrofitting measures which prevent the activation of local mechanisms and ensure a global structural response during earthquakes.

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