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Pushover analysis for upgrading of existing residential masonry building

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ABSTRACT

Masonry is common used for classic building construction, but is known for its seismic vulnerability. The existing regulations in Serbia (Eurocode) demand control of the behaviour of masonry buildings exposed to seismic actions. For this it is most convenient to use, for engineering purposes, non-linear static (pushover) analysis. With the aim of increasing knowledge about the seismic behavior of different structural configurations created by upgrading an existing residential two-story masonry building, a comparative study was conducted on four different proposed models. This paper presents the most significant results of pushover analyses performed on a spatial model of the structure of the existing building that needs to be upgraded and on spatial models of the structures with an additional floor level and different positioning of vertical confining elements while evaluating their seismic capacity in compliance with the regulations. Two different lateral load distributions are assumed in computation since the appropriate lateral load profile is not always obvious. Based on various approaches, there are different identified responses.

KEYWORDS

Masonry building, Storey upgrading, Pushover analysis, Seismic behavior

1. INTRODUCTION

Masonry construction is common, and it has gained popularity mainly due to its low cost, widespread geographical availability, thermal insulation, protection from fire, durability, low maintenance cost, and it is easy to construct. Masonry has good compressive strength, thus the structures will behave well as long as the loads are vertical, but when a horizontal inertial earthquake forces act, they start to develop shear and flexural stresses. Masonry is a non-homogeneous, non-isotropic material, with a mechanical behavior dominated by the non-linear phase, characterized by negligible strength and brittleness in tension, and dissipative with softening behavior in compression. To increase the number of standardized economic solutions of masonry structures, further research is necessary into the problems that may occur during their construction and exploit. At the same time, based on the previous facts, it is necessary to consider masonry structures as vulnerable to earthquakes[1].

An assessment of the seismic vulnerability of masonry structures is a complex task that must be performed by engineers. Therefore, it is necessary to make a balance between sufficient accuracy and simplicity and availability of the assessment for frequent use in practice. For this purpose, it is essential to know the characteristics of the structure being designed and the relevant actions on it, as well as the properties of the materials that will be used for construction. All the unreliability of the assessment and input data can create unfavorable solutions and lead to undesirable consequences [2].

The regulations for the design of masonry structures under static and dynamic actions in Serbia have been significantly improved with the introduction of Eurocodes. An assessment of the seismic resistance of the common masonry structures is conducted in accordance with provisions from the regulation: SRPS EN 1996-1-1 [3], SRPS EN 1998-1 [4], SRPS EN 1998-1/NA [5], and SRPS EN 1998-3 [6].

The seismic vulnerability of masonry structures is rarely assessed by linear elastic analysis procedures. Non-linear dynamic analysis methods represent the most reliable tool. Nevertheless, they are very complex and require a great amount of computational resources and time, and further research efforts are still needed before they can be confidently used in standard design. Therefore, pushover procedures have been increasingly recognized as effective tools in seismic design and vulnerability assessment: they provide information on both the strength and ductility of the structure while preserving the simplicity of static analysis. Here a single degree of freedom (SDOF) system was derived to represent the multi-degree of freedom (MDOF) structure via an equivalent or "substitute" structure. However, the simplicity of these procedures is paid with a series of limitations that may restrict their application only to some classes of structures. The main outcome of pushover procedures is the curve relating the displacement of a certain controlling point to the resultant of a predefined horizontal distributed force applied to the structure. This curve, representing the seismic capacity of the structure, is then compared with the seismic demand, expressed in terms of response spectrum [7, 8].

Two basic things define different pushover procedures, one being the choice of the load pattern and the second simplification model of the pushover curve for design purposes. In pushover procedures the magnitude of structural loading is incrementally increased following a certain predefined pattern, thus causing weak links and developing failure modes. Pushover analysis is basically an attempt to evaluate the real seismic capacity of the structure, and it is effective for performance-based design. A drawback of the method is that for masonry structures, until now, the best pattern of loads is not yet determined. Additionally, it gives a time-independent displacement shape. The advantage of this procedure is that it is able to locate the most vulnerable parts of the structure [8].

The use of an equivalent static, simplified non-linear (pushover) method for seismic assessment of masonry structures was introduced in 1978 by Tomaževic(SFR Yugoslavia, today Republic of Slovenia) [9]. Such a method has undergone several refinements through the years and firstly was formulated anInvariant-force Pushover Analysis (IPA).This method considered that pushover analyses with invariant lateral force distribution cannot detect changes caused in non-linear dynamic characteristics due to the evolution of damage in the structure. More advanced pushover analysis like Modal Pushover Analysis (MPA) was developed based on lumped mass systems like frame structures. Unlike previous methods,the MPA permits the consideration of higher modes in pushover analysis and asymmetrical structures in the layout plan. One of the limitations of this method is that the sequence of damage development cannot be directly controlled, since the final deformed shape can be represented only by the superposition of the deformed shapes from each mode. Almost at the same time as the MPA, Adaptive Pushover Analysis (APA) was developed. The APA can represent the development of the damage during the analysis by updating the lateral force distribution pattern as damage propagates. This method considers inelasticity at the current step and updates the lateral load distribution accordingly. SRPS EN1998-1 suggests the application of the N2 method, based on the combination of pushover analysis with the capacity spectrum approach. The N2 method correlates the displacement capacity of the structure to the displacement demand of the expected earthquake. At first, the method was introduced for symmetrical structures, for which good performance was observed. Additionally, over time, the N2 method has extended to asymmetrical frame structures, structures irregular in layout plan, and structures irregular both in plan and height. The N2 introduced the impact of torsion with higher modes as well, and is relatively easy to use, thus it is more economical in relation to the use of non-linear dynamic analysis[10-12].

It has been observed that under the action of moderate to severe earthquakes the masonry structures performed the worst, causing the largest loss of lives as well as properties. Thus, to save the people from the collapse of such buildings during an earthquake it is required to make them earthquake resistant. For existing buildings, seismic vulnerability evaluation is needed, along with possible retrofitting [13]. Also, with an increase of population in cities, there is a constant need for new residential spaces. As a more economical solution than the construction of new buildings, upgrading existing buildings by adding one or more floor levels is often conducted. Such new structures must meet the same seismic resistance regulations as the existing ones. On the other hand, there are currently insufficient guidelines for modelling uncommon masonry structural configurations [2]. For these purposes, researchers are evaluating different types of masonry structures that require retrofitting and upgrading. By using pushover analysis, Manojlović et al. showed the possibilities of seismic retrofitting and upgrading an existing grammar school masonry building in Novi Sad, Serbia [14]. Galasco et al. showed the three-dimensional model of an old masonry building assembled with frame-type macro-element models of the walls and orthotropic membrane elements to represent the mechanical behavior of flexible timber floors. This modelling approach, although very effective in representing the actual behavior, does not allow to use of common simplifications such as rigid floor motion [15]. Ademović et al. discussed the behavior of a typical masonry building in Bosnia and Herzegovina built in the 1950s without any seismic guidelines [8]. Bocciarelli and Barbieri proposed a numerical procedure for pushover analysis of an old masonry tower with displacement control [1].A similar discussion was provided by Shehu[16]. Giordano et al. investigated the seismic response of plan irregular (asymmetric) two-storey masonry building structure to evaluate the magnitude of torsional coupling and the applicability of 3D pushover analysis for assessing the behavior under earthquakes[17].

This paper shows a comparative study for upgrading of existing residential two-storey masonry building by adding one additional floor level on four proposed models. The results of pushover analyses performed on proposed models were evaluated with the respect to the regulations. In the computations, two different lateral load distributions were assumed since the appropriate lateral loads profile is not always obvious. Based on various considered modelling approaches, different responses of the structures were identified.

2. PUSHOVER ANALYSIS ACCORDING TO EUROCODE 8-1

According to SRPS EN 1998-1, a pushover analysis must be performed with two lateral load patterns. A load distribution corresponds to the fundamental mode shape and a uniform distribution proportional to storey masses. Classical steps of the pushover analysis are:

- Modelling of the structure with concentrated storey masses with elastic behavior.
- Determination of the fundamental period of vibration.
- Determination of the fundamental mode shape (Eigen vectors).
- Determination of the modal mass coefficient for the first natural mode.
- Determination of the lateral displacements at each level for the first natural mode.
- Determination of seismic forces at each level for the first natural mode.
- Transformation of the MDOF system to an equivalent SDOF with an equivalent mass m^* as formulated in (1).

$$m^* = \sum m_i \cdot \Phi_i = \sum \bar{F}_i \tag{1}$$

- Determination of the modal participation factor of the first natural mode according to (2).

$$\Gamma = \frac{m^*}{\sum m_i \cdot \Phi_i^2} = \frac{\sum \bar{F}_i}{\sum \left(\frac{\bar{F}_i^2}{m_i} \right)} \tag{2}$$

- Determination of the force F^* and the displacement d^* of the equivalent SDOF represented in eq. (3, 4).

$$F^* = \frac{F_b}{\Gamma} \tag{3}$$

$$d^* = \frac{d_n}{\Gamma} \tag{4}$$

In (3, 4) F_b is the base shear force and d_n the control node displacement of the MDOF system.

- Determination of the idealized bilinear force-displacement relationship as shown in Figure 1. The yield force F_y^*, which represents also the ultimate strength of the equivalent SDOF system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the equivalent SDOF system is determined in such a way that the areas under the actual and the equivalent SDOF system force displacement curves are equal. Based on this assumption, the yield displacement of the equivalent SDOF system d_y^* is given in (5).

$$d_y^* = 2 \cdot \left(d_m^* - \frac{E_m^*}{F_y^*} \right) \tag{5}$$

In (5) E_m^* is the actual deformation energy up to the formation of the plastic mechanism. Figure 1 shows the principle of energy idealization.

- Determination of the period of the equivalent SDOF system T^* as in (6).

$$T^* = 2 \cdot \pi \cdot \sqrt{\frac{m^* \cdot d_y^*}{F_y^*}} \tag{6}$$

- Determination of the target displacement for the equivalent SDOF system is given in (7).

$$d_{et}^{*} = S_e\left(T^{*}\right)\cdot\left(\frac{T^{*}}{2\cdot\pi}\right)^2 \tag{7}$$

$S_e\left(T^{*}\right)$ is the elastic acceleration response spectrum at the period T^{*}. Determination of the target displacement d_t^{*} for structures in the short period range is formulated in (8) for the elastic response and (9) for the non-linear response, while $T^{*} < T_c$. For structures in the medium and long period ranges, target displacement is given in (10), while $T^{*} \geq T_c$. q_u is the ratio between the acceleration in the structure with unlimited elastic behavior $S_e\left(T^{*}\right)$ and in the structure with limited strength F_y^{*}/m^{*}. T_c is the border between short and medium vibration periods.

$$If\ \frac{F_y^{*}}{m^{*}} \geq S_e\left(T^{*}\right) \rightarrow\ d_t^{*} = d_{et}^{*} \tag{8}$$

$$If\ \frac{F_y^{*}}{m^{*}} < S_e\left(T^{*}\right) \rightarrow\ d_t^{*} = \frac{d_{et}^{*}}{q_u}\cdot\left(1+\left(q_u-1\right)\cdot\frac{T_c}{T^{*}}\right) \geq d_{et}^{*};\ q_u = \frac{S_e\left(T^{*}\right)\cdot m^{*}}{F_y^{*}} \tag{9}$$

$$d_t^{*} = d_{et}^{*} \tag{10}$$

- The target displacement of the MDOF system is presented in (11).

$$d_t = \Gamma\cdot d_t^{*} \tag{11}$$

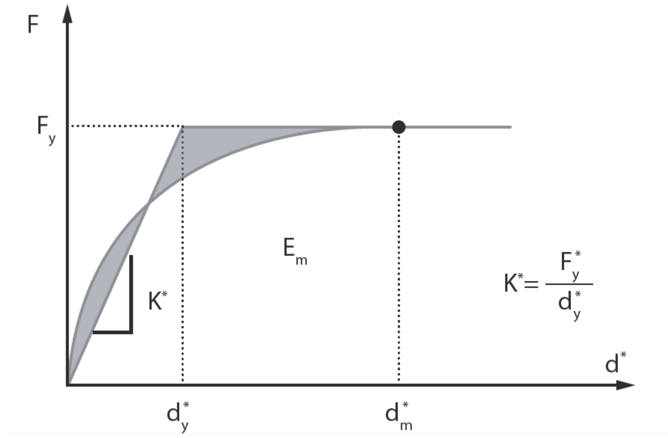

Figure 1: Linearization of the capacity curve[18]

3. CASE STUDY

To evaluate the most suitable structural solution, in terms of seismic vulnerability, for the upgrade of an existing residential two-storey masonry building, four different models of upgrading were made and their responses were observed, as well as the behavior of the existing building. Characteristic floor layout and three-dimensional representations of models are shown in Figure 2 and marked as:

- M0 - model of the structure of the existing building, as an unreinforced masonry building, with the found mechanical and geometric characteristics of load-bearing structural elements.
- M1 - model of the structure of the upgraded existing building with an additional floor modelled like the lower floors (unreinforced masonry), only with horizontal confining elements along the edges of the slabs, without vertical confining elements.
- M2 - model of the structure of the upgraded existing building with an additional floor modelled like confined masonry with horizontal confining elements along the edges of the slabs and vertical confining elements at the ends and crossings of the walls and at appropriate distances, while lower floors remain unchanged like unreinforced masonry.
- M3 - model of the structure of the upgraded existing building with an additional floor modelled like confined masonry with horizontal confining elements along the edges of the slabs and vertical confining elements that extend from the foundation to the top of the building only on the shorter façade walls of the building.

- M4 - model of the structure of the upgraded existing building with an additional floor modelled like confined masonry with horizontal confining elements along the edges of the slabs and vertical confining elements that extend from the foundation to the top of the building located in characteristic places on all façade walls of the building.

For modelling of the building structure and proposed upgraded solutions, the software AmQuake was used with the finite element method for numerical calculation and common macro elements. Additionally, the software used the equivalent frame approach for the pushover analysis which considers all modelled line structural elements with their inelastic behavior. All models have openings in walls in accordance with architectural design, and around them software generated new line macro elements for parapets and spandrels. The assumption of rigid floor motion was introduced in all models. The AmQuake uses pushover analysis and determines the target displacements in accordance with the provisions of regulations [19]. The obtained target displacements were compared with the corresponding capacities.

Although SRPS EN 1998-1 requires the use of mean values of the material properties, in this study the characteristic values were used, which provided a certain safety to the results of the pushover analysis. In the analysis, the stiffness of cracked sections was taken as half of the stiffness value of uncracked sections. Soil type C, acceleration $a_{gR}= a_g=$ 0.20g,and building importance class II represented the seismic input. As SRPS EN 1998-1demands, two different load patterns of the seismic force were applied to all models for the pushover analysis, triangular and uniformly distributed lateral force in positive directions of the X and Y axis[4]. Additionally, in both these directions were added eccentricities equal to +0.05 times the building dimension in the plane perpendicular to the direction of the seismic force for the purpose of creating accidental torsional effects.

Pushover analysis was conducted with consideration of the ultimate limit state (ULS) and damage limitation state (DLS). By SRPS EN 1998-3, for the masonry and reinforced concrete elements were taken drift limit for bending failure equal to 0.008 and drift limit for shear failure equal to 0.004[6]. The analysis included only the stiffness of the load-bearing walls, while the stiffness of the partition walls was not taken into account, only their mass.

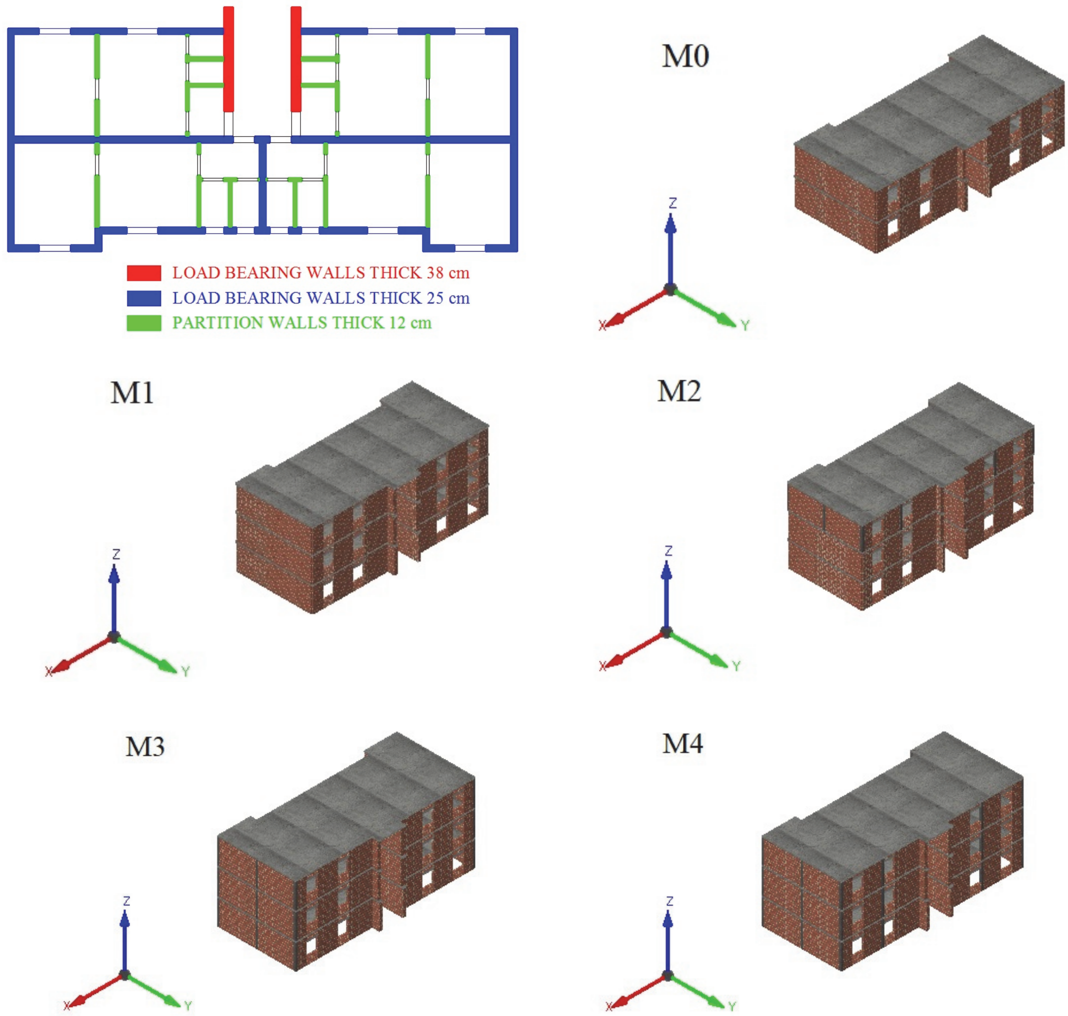

Figure 2: Characteristic floor layout and three-dimensional representations of models for the existing building M0, and upgrading solutions M1-4

3.1. Materials and loads of the existing building

Representing masonry building is situated in Serbia and it is relatively new. It consists of a high ground floor and one upper floor, with a gross floor area of 199.55 m^2 and a characteristic floor layout as shown in Figure 2. Each floor level consists of four residential units with similar gross areas. Vertical communication inside the building is provided by the central staircase area. Storey height is equal to 300 cm and the clear height is equal to 280 cm. Structural walls were constructed using clay blocks without reinforcement, and floor slabs were made with reinforced concrete (RC). During inspections of building design data, no vertical confining elements were observed, only horizontal confining RC elements were found at floor levels and lintels made with reinforced concrete.

The façade walls of the building are with a thickness of 25 cm as internal load-bearing walls. The walls around the staircase are with a thickness of 38 cm and the partition walls are 12 cm thick. All walls are made of porotherm clay blocks and dry-fix mort. Characteristic compressive strength calculated based on SRPS EN 1996-1-1 is f_k = 3.87 MPa for bearing walls thickness of 25 and 38 cm. Characteristic initial shear strength of masonry under zero compressive stress was adopted asf_{vk0}= 0.30 MPa, whereas for the limit value of characteristic masonry shear strength f_{vlt} =0.52 MPa was used. Furthermore, the short-term secant modulus of elasticity of masonry was determined as $E = 1000 \cdot f_k$ = 3.869GPa. For the shear modulus of masonry, a calculated value of G =1.548GPa was adopted. Horizontal confining elements and lintels were made with concrete class C16/20 (f_{ck}= 16 MPa) along with reinforcement B500 (f_{yk}= 500 MPa). Concrete elements have a rectangular cross-section of 25/20 cm, and they are reinforced with four 12 mm diameter longitudinal bars and 6 mm diameter stirrups spaced at 15 cm. Solid concrete slabs are designed on all floor levels, including the roof.

The existing building was designed with self-weight applied to every floor structure amounted to 4.0 kN/m^2, an additional dead load applied at floors amounted to 1.5 kN/m^2, and applied variable load at the floor (except the roof) amounted to 2.5 kN/m^2.The roof floor structure was designed with self-weight, additional dead load, and the snow load equal to 5.0 kN/m^2, 0.5 kN/m^2, and 1.0 kN/m^2, respectively.

3.2. Materials and loads of the upgraded floor level

To upgrade the existing building, the considered models used walls with the same mechanical and geometric characteristics as the walls on the lower floors. All added RC structural elements are designed from concrete classC25/30 (f_{ck}= 25 MPa). Along the edges of the slabs, horizontal confining elements with a rectangular cross-section of 25/20 cm were designed and reinforced with four 14 mm diameter longitudinal bars and 6 mm diameter stirrups spaced at 15 cm. The lintels have the same characteristics as the horizontal confining elements. Vertical confining elements were designed with a square cross-section of 25/25 cm at the ends and crossings of the walls and at appropriate distances. They were also reinforced with four 14 mm diameter longitudinal bars and 6 mm diameter stirrups spaced at 15 cm.

The additional floor level was designed with a new RC roof slab and with applied self-weight, self-weight of the roof structure, additional dead load, and the snow load to 3.0 kN/m^2, 2.5 kN/m^2, 0.5 kN/m^2, and 1.0 kN/m^2, respectively.

4. RESULTS OF THE STUDY

Pushover analysis was performed with both types of elastic spectrum. This paper shows only results of the analysis with type 1 spectrum, as authoritative for the given soil acceleration. For target displacements, the N2 method was used as part of regulations in the software AmQuake[19]. Tables 1-2 show the most important parameters of the analysis for the uniform and the triangular distributions of the seismic force in X and Y positive directions: yield force (F_y) of the idealized bilinear pushover curve for the MDOF system, target top floor displacements for the ULS ($d_{t,ULS}$) and DLS ($d_{t,DLS}$), the ULS (d_t) and DLS ($d_{c,DLS}$) capacities for the top floor displacements, calculated maximum inter-storey drift (*RShift*), DLS inter-storey drift criterion (*RShift* criterion), period of vibration of the idealized structural system with SDOF (T^*), capacity ductility factor μ and yield displacement (d_y) of the idealized bilinear pushover curve for the MDOF system.

Table 1: Most important results of the pushover analysis for building models with uniform distributed load

Model	M0		M1		M2		M3		M4	
Direction	+X	+Y	+X	+Y	+X	+Y	+X	+Y	+X	+Y
F_y[kN]	1644.44	1503.91	1721.32	1380.19	2073.71	2249.21	1776.05	2493.19	2072.77	2553.65
$d_{t,DLS}$[mm]	1.278	1.105	3.600	2.766	1.745	1.715	3.909	2.881	4.230	2.872
$d_{c,DLS}$[mm]	4.956	5.097	15.597	17.197	7.600	6.601	16.412	10.390	15.400	10.564
$d_{t,ULS}$[mm]	4.894	4.928	17.925	16.839	7.399	6.230	18.755	10.313	17.843	9.951
d_t[mm]	4.956	5.097	15.372	17.197	7.600	6.601	15.812	10.390	14.600	10.564
RShift	0.0009	0.0013	0.0044	0.0032	0.0020	0.0015	0.0041	0.0020	0.0044	0.0022
RShift criterion	0.177	0.259	0.872	0.642	0.392	0.309	0.822	0.394	0.876	0.431

T^{*}[s]	0.140	0.133	0.217	0.192	0.161	0.160	0.226	0.196	0.234	0.195
μ	2.118	2.674	3.402	6.032	2.568	2.084	3.174	1.989	2.334	1.987
d_y[mm]	2.340	1.906	4.518	2.851	2.960	3.168	4.982	5.223	6.256	5.317

Table 2: Most important results of the pushover analysis for building models with triangular load

Model	M0		M1		M2		M3		M4	
Direction	+X	+Y	+X	+Y	+X	+Y	+X	+Y	+X	+Y
F_y[kN]	1231.14	1165.31	1311.20	1046.49	1910.89	2010.59	1319.58	2118.42	2047.59	2203.59
$d_{t,DLS}$[mm]	0.942	1.071	4.447	8.030	2.442	2.063	4.313	3.173	4.733	3.089
$d_{c,DLS}$[mm]	4.337	6.097	15.542	20.192	10.462	7.759	21.018	12.190	19.072	11.425
$d_{t,ULS}$[mm]	4.179	5.741	24.005	34.250	10.323	7.677	23.628	12.130	18.632	11.341
d_t[mm]	4.337	6.097	15.392	20.042	10.462	7.759	20.773	12.190	19.072	11.425
RShift	0.0011	0.0017	0.0040	0.0040	0.0024	0.0016	0.0041	0.0021	0.0033	0.0019
RShift criterion	0.224	0.336	0.800	0.789	0.488	0.318	0.810	0.413	0.658	0.387
T^{*}[s]	0.120	0.127	0.223	0.275	0.172	0.161	0.218	0.190	0.230	0.189
μ	2.617	3.608	3.274	3.504	2.565	2.063	4.565	2.201	2.500	2.061
d_y[mm]	1.657	1.690	4.702	5.720	4.078	3.761	4.550	5.538	7.630	5.543

According to the presented results, two models meet the ULS criteria, the model of the existing building M0 and the combined model of the upgraded building M2 where the lower floors are left with unreinforced masonry (URM) and the added floor contains horizontal and vertical confining elements (CM). In the case of the DLS criteria, it was found to be irrelevant for the proposed models, as all target parameters meet the criteria.

Yield force (F_y) is larger in the X direction under uniform load for the URM models M0 and M1, while for the models M2-4 with CM yield force is larger in the Y direction under triangular load. The smallest difference in the magnitude of the horizontal seismic force when the load is uniform or triangular is for M2 which is the only model of upgrading that meet ULS criteria, while the largest difference between the horizontal forces occurs on M3.

From Figures 3-7 it can be seen that for all models except M1 the DLS displacements ($d_{t,DLS}$) are always on the parts of the pushover curves corresponding to the linear elastic structural response, which is an expected result. Only for model M1 $d_{t,DLS}$ was observed on part of the pushover curve related to the non-linear response.

The most critical parameter of the analysis is ULS top floor displacement ($d_{t,ULS}$) that can be expected for the given structure and seismic demand, and it must be smaller than top floor displacement when the structure starts to collapse (d_t) or when the base shear force drops below 80% of its maximal value. As stated, this criteria was met only by two models, M0 and M2. In terms of safety margin for the ULS criteria, defined as the percentile ratio between the target and the capacity values as presented in (12), model M1 performed with the lowest safety of -70,89% for Y direction and triangular load, then models M4, M3, M2, M0 with their the most unfavourable safety of -22,21% (X, uniform load), -18,61% (X, uniform load), 1,05% (Y, triangular load), 1,25% (Y, triangular load), respectively. These, the most unfavourable displacements for each model are shown in Figures 3-7 with the pushover force-displacement curve and idealized bilinear force-displacement diagram. For URM model M1 it is expected to perform the worst in terms of displacements in the Y direction which is shorter and has less stiffness and under a triangular load pattern that accumulates higher top floor displacements. Similar behavior showed model M2 which has only vertical confining elements on top floor level and lower levels are URM, thus it has an increase in stiffness, decrease in inter-storey drift, and overall displacements values. Different behavior showed models M3-4 with vertical confining elements from the top floor to the foundation on the façade, and they performed the worst in the X direction under a uniform load pattern, thus indicating the formation of hinges in the vertical confining elements. This change in unfavourable conditions is probably due to inserted vertical confining elements increasing more stiffness in the Y than X direction.

$$ULS\ safety\ margin = \frac{d_t - d_{t,ULS}}{d_t} \cdot 100\% \qquad (12)$$

RShift presents calculated maximum inter-storey drift from all storey's *deflection/storey height* and it is compared with DLS inter-storey drift criterion (*Rshift/Damage limit*). For three-storey models, the highest *RShift* has URM model M1. The introduction of vertical confining elements from the top floor to the foundation on the façade had lowered *RShift*, but only significantly in a shorter Y direction. Combined model M2 performed the best in terms of inter-storey drift. This may be due to the fact that in SRPS EN 1998-3 the same values of drift limits apply for all masonry structures, even though they are explicitly defined for the URM ones.

Periods of vibration of the idealized structural system with SDOF (T^{*}) for proposed models are similar for different directions and higher in the longer X direction as expected, only M2 with triangular load is an exception. All periods have lower values than period with constant acceleration in the elastic response spectrum (T_c = 0.6 s for soil type C), thus all models are in the short period range.

Capacity ductility factor μ for ULS is determined as the ratio between capacity displacement (d_t) and yield displacement (d_y), and given values in tables 1-2 were calculated manually since AmQuake provides target ductility factor values calculated with target displacement. For models M0 and M2 that meet the ULS criteria with high non-linear response, there is a low ductility reserve which makes these models sensitive to seismic action, as shown on Figures 3 and 5.

On the left side of Figures 3-7 is shown the deformation of models with damage factor *CF* (confidence factor), which is a parameter indicating how much the element response deviates from the elastic one. It ranges from 0 for the fully undamaged to 1 for the fully damaged conditions.

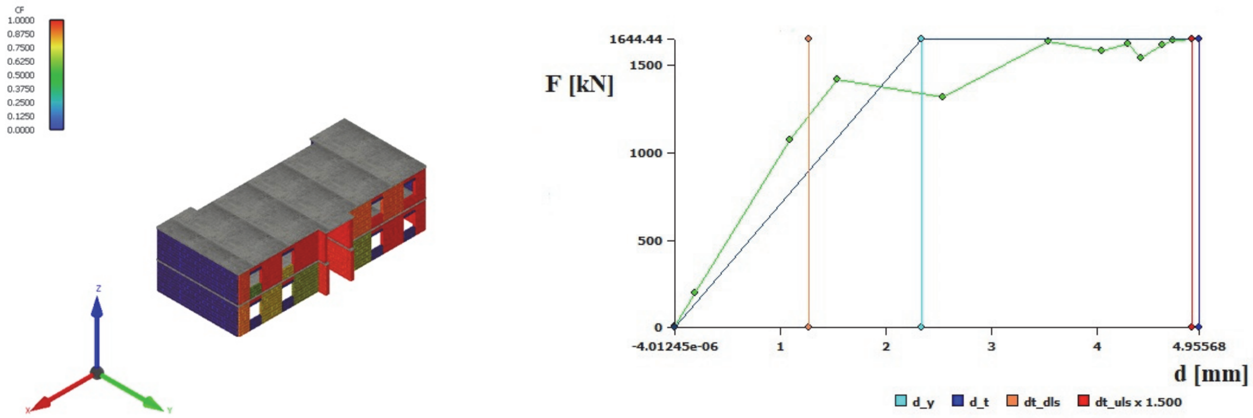

Figure 3: (left) Damage factor *CF*, (right) pushover force-displacement curve with idealized bilinear force-displacement diagram for the model M0, +X direction and uniformly distributed load

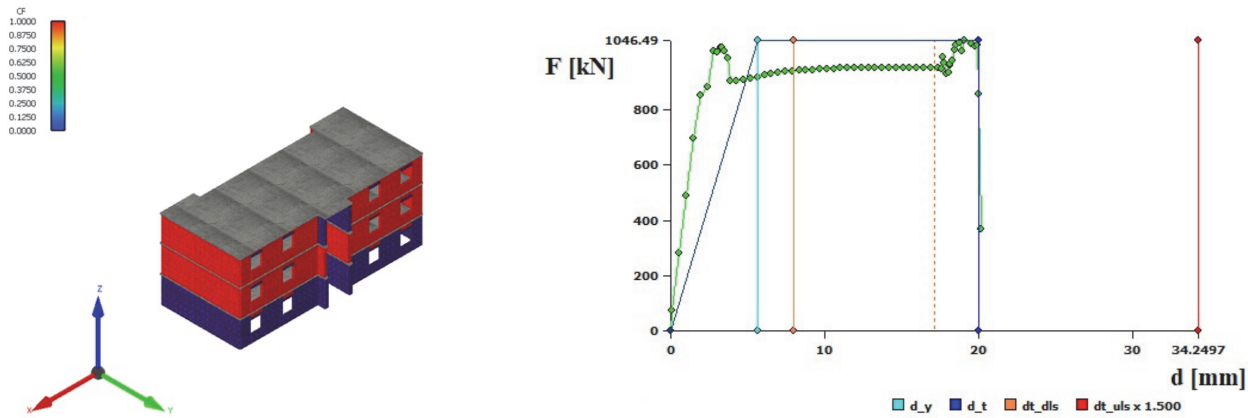

Figure 4: (left) Damage factor *CF*, (right) pushover force-displacement curve with idealized bilinear force-displacement diagram for the model M1, +Y direction and triangular load

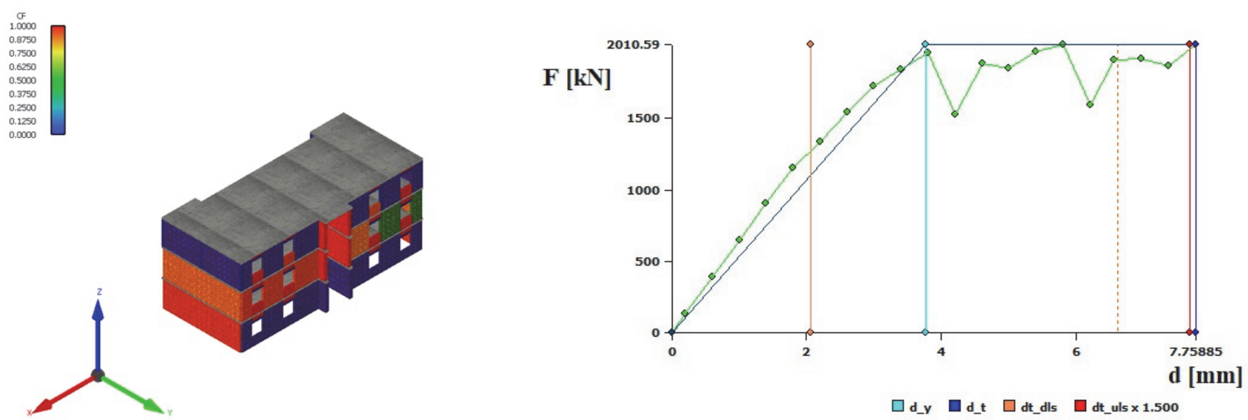

Figure 5: (left) Damage factor *CF*, (right) pushover force-displacement curve with idealized bilinear force-displacement diagram for the model M2, +Y direction and triangular load

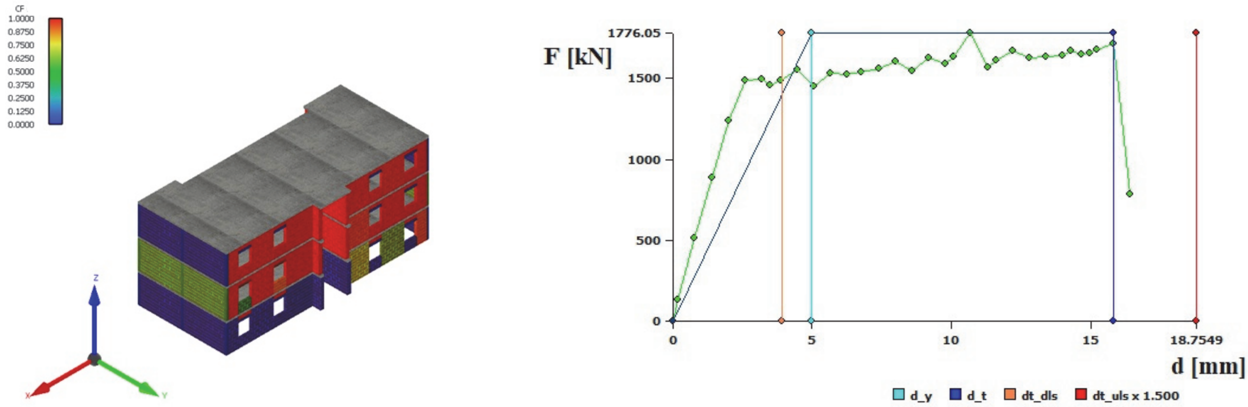

Figure 6: (left) Damage factor *CF*, (right) pushover force-displacement curve with idealized bilinear force-displacement diagram for the model M3, +X direction and uniformly distributed load

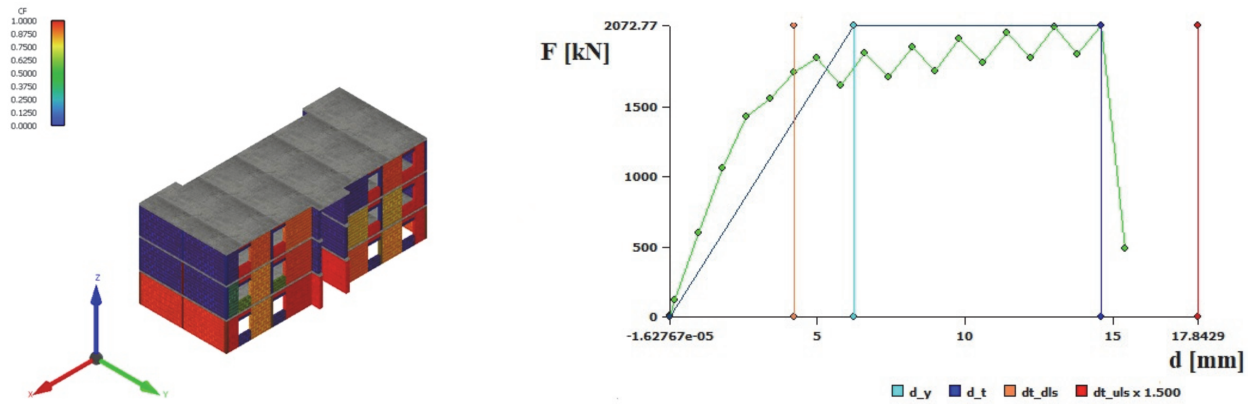

Figure 7: (left) Damage factor *CF*, (right) pushover force-displacement curve with idealized bilinear force-displacement diagram for the model M4, +X direction and uniformly distributed load

5. CONCLUSIONS

In this paper the seismic vulnerability of an existing two-story unreinforced masonry building was analyzed, and models for its upgrade with another floor level were proposed, by using pushover analysis. The existing building showed good properties for the effect of horizontal forces from earthquakes. It was analyzed four upgraded models of existing building:

1. model without vertical confining elements,
2. model with vertical confining elements only in the added floor level,
3. model with vertical confining elements along the shorter façade walls and along the entire height of the building,
4. model with vertical confining elements along the entire height of the building along all façade walls.

The ULS parameters from earthquake using pushover analysis, was satisfied only by a second model with vertical confining elements only in the added floor level. The model without vertical confining elements performed the worst.

Current regulations demand the use of two different lateral load patterns in pushover analysis without a detailed explanation of each pattern's purpose. This paper confirmed that, as during analysis models showed different behaviors with uniformly distributed or triangular loads depending on proposed confining. While using software AmQuake for pushover analysis capacity ductility factor should be calculated manually.

Seismic response of structure is in a tight connection with confining, and it has beneficial effects especially in a case of the shear capacity. Positioning of confining elements has a key role for evaluation of seismic vulnerability, and ductility capacity of the masonry structures. Further research on this topic must include models with retrofitting solutions that are more common in practice, such as strengthening of the existing walls or installing vertical confining elements along the entire height of the building and in all load-bearing walls. There is a need for guiding the engineers toward proper and better modelling of masonry structures.

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