RESPONSE OF EXISTING AND UPGRADED MASONRY CHURCH TOWER CONSIDERING THE CHANGE IN SEISMIC HAZARD

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Summary: *A typical masonry church tower was analyzed by means of pushover analysis performed on a spatial finite element model consisting of macro-elements. For two different levels of seismic hazard, the differences in the responses of the existing and upgraded tower structures were examined and discussed. Special attention has been given to the analysis of displacement demands and capacities corresponding to the damage limitation and ultimate limit states, with the respect of the Eurocode 8 provisions. Additionally, the maximum possible values of design ground accelerations were determined for the considered cases in order to investigate the reserve in structural capacities.*

Keywords: *masonry church tower, seismic hazard, pushover analysis, Eurocode*

1. INTRODUCTION

An appropriate assessment of the response of existing historical sacral masonry buildings during earthquakes is a great challenge faced by engineers worldwide. It requires a high level of knowledge on all relevant material and structural properties. Any lack of relevant data, combined with the uncertainties related to the applied seismic input, usually leads to questionable results. Additionally, structural modelling approach usually has a crucial impact on the output. Thus, by taking into account the fact that sacral masonry buildings are very common and important, it is obvious that their assessment needs to be conducted with a special caution.

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There is a progress in code provisions related to the design and analysis of new masonry buildings exposed to both static and dynamic loads, e.g. Eurocode 6 (Part 1-1 [1]) and Eurocode 8 (Part 1 [2]). Also, the issues related to existing masonry buildings are nowadays covered in a fairly good manner, e.g. with provisions of Eurocode 8 (Part 3 [3]). On the other hand, in order to properly assess the behavior of an existing masonry building exposed to the seismic action, at least a nonlinear static (pushover) analysis needs to be performed, which is in accordance with Eurocode 8. Nevertheless, not enough related code guidelines can be found, especially when it comes to structural modelling. It should be noted that several beneficial modelling and analysis software tools have been recently developed based on the finite element method, experimentally obtained data, and observations made on buildings damaged by earthquakes. This fact implies that in the future the most commonly used "hand" calculations and checks shall be substituted with more sophisticated and reliable approaches.
In this paper a typical masonry church tower was analyzed by pushover analysis performed on a spatial finite element model. For different levels of seismic hazard, the differences in responses of the existing and upgraded structure were examined and discussed.

2. DESCRIPTION OF STRUCTURES AND SEISMIC INPUT

The masonry church tower analysed in this paper was considered in two variants, i.e. in its existing and upgraded conditions (Fig. 1). The plan view and all relevant elevation and cross-sectional dimensions are presented in Fig. 2.

Figure 1 – View of existing and upgraded church tower

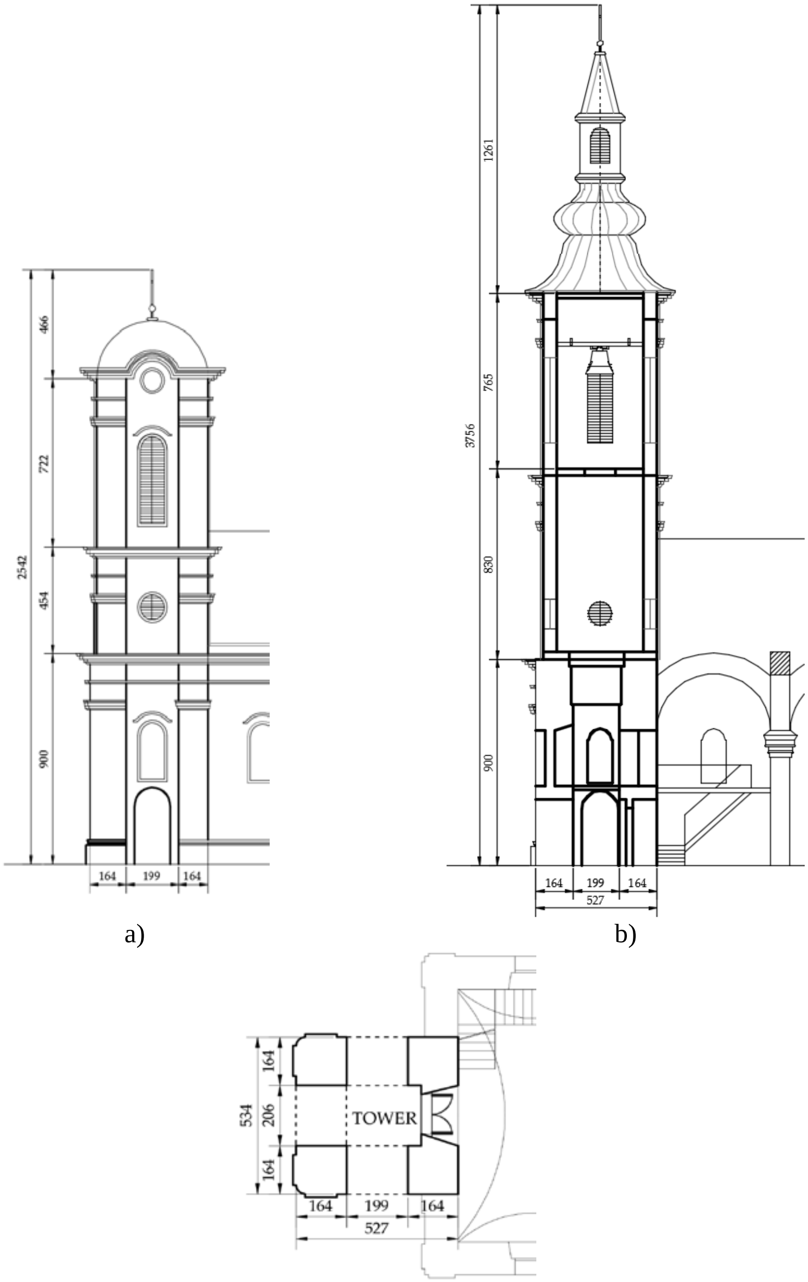

Figure 2 – Elevation and cross-section of (a) existing and (b) upgraded structures, and (c) plan view of the lowest storey

Besides self-weight of the tower structure, the weight of the bell and its supporting structure was applied at the top of the tower, and variable load amounting to 1.0 kN/m^2 was applied at floors where applicable. The snow load on the roof was neglected due to the roof slope. The main properties of the structures are briefly summarized below.
At the lower part of the tower, i.e. up to approximately 9.15 m, wall thickness amounts to 164 cm for both considered variants. At the rest of the height, up to approximately 21.20 and 25.0 m for the existing and upgraded tower (respectively), wall thickness amounts to 60 cm.
Masonry walls are designed in solid clay bricks which belong to Group 1 (f_b = 10 and 15 MPa for existing and new masonry, respectively) with the general purpose mortars M2 (existing masonry) and M5 (new masonry) with f_m amounting to 2 and 5 MPa, respectively. According to Eurocode 6, characteristic compressive strength (f_k) of the existing and new masonry walls amounts to 3.5 and 6.0 MPa, respectively, with corresponding moduli of elasticity (E) equal to 3500 and 6000 MPa, and corresponding shear moduli (G) equal to 1400 and 2400 MPa. In the case of horizontal confining elements, concretes C16/20 (f_{ck} = 16 MPa) and C20/25 (f_{ck} = 20 MPa) were used in the case of the existing and upgraded structures (respectively), along with steel S240 and S500 (f_y amounts to 240 and 500 MPa). Confining elements have rectangular cross-section 60/30 cm, and they are reinforced with eight 12 mm diameter longitudinal bars and 8 mm diameter stirrups spaced at 25 cm. In the lower part of the tower structure timber floor is present, whereas at the upper part one-way masonry-reinforced concrete composite slabs of 20 cm depth are designed by using the concrete C20/25.
Eurocode 8 type 2 spectrum for soil type C with a_{gR} equal to 0.05 and 0.10 g (old and new seismic hazard maps, respectively) represented the seismic input. In both cases, importance class was assumed to be III (γ_I = 1.2). Therefore, a_g amounted to 0.06 and 0.12 g for considered levels of hazard. It should be noted that these values were used for the verification of the ultimate limit state (ULS), whereas for the damage limitation state (DLS) verification lower values of a_g were considered, amounting to 0.024 and 0.048 for low and high seismic hazards, respectively.

3. STRUCTURAL MODELING AND ANALYSIS PARAMETERS

Macro-element models are most commonly used when the finite element method is applied on masonry structures. The equivalent frame approach was used for the purpose of the analysis presented herein. More details on the equivalent frame models can be found elsewhere (see e.g. [4], [5], [6], [7]).
In pushover analysis, in order to achieve the results which are on the safe side, the characteristic values of material properties were taken into account, even though Eurocode 8 suggests that the mean values should be used. In addition, the reduced stiffness of cracked sections was taken into account, assumed to be equal to one-half of the stiffness of corresponding homogenous sections. Where applicable, floor slabs were modelled as rigid diaphragms. All openings and horizontal confining elements in walls were included in the models as well.
The capacity of individual members in terms of drift limits was considered according to the provisions of the Part 3 of Eurocode 8. In the case of the ULS, which is roughly equivalent to the limit state of significant damage (SD) defined in the Part 3 of Eurocode

8, the drift limits taken into account amounted to 0.80% and 0.40% for flexure and shear, respectively. Both values correspond to the unreinforced masonry, which is actually the only case explicitly considered in the Part 3 of Eurocode 8. In the case of the DLS which, as expected, found to be the irrelevant limit state for the considered structure, the assessment was conducted with the respect to the inter-storey drift limit of 0.50%, as prescribed in the Part 1 of Eurocode 8.
For both considered structural models, pushover analysis was performed by assuming the inversed triangular pattern of lateral force distribution since it produces relevant results. In terms of the church plan, only the transverse direction was taken into account since it can be considered as the "weak" one. Additionally, accidental torsional effects were neglected.

4. RESULTS OF THE STUDY

Target displacements were determined by using the N2 method (for more details see [8] and [9]). For clarity, all relevant and representative results are shown in Table 1, which contains the following data: yield force F_y and displacement d_y of the idealized bilinear pushover curve, target displacements for the DLS ($d_{t,DLS}$) and ULS ($d_{t,ULS}$), and capacities for the DLS ($d_{c,DLS}$) and ULS ($d_{c,ULS}$). Note that all quantities were obtained for MDOF system. Additionally, capacities in terms of the maximum possible a_g values for the ULS are shown as well.

Table 1 - The most important results obtained from pushover analysis

Building	**existing**		**upgraded**	
a_g	**0.06 g**	**0.12 g**	**0.06 g**	**0.12 g**
F_y **[kN]**	375		464	
d_y **[mm]**	15.7		24.0	
$d_{t,DLS}$ **[mm]**	5.8	11.6	7.0	14.1
$d_{t,ULS}$ **[mm]**	14.5	29.1	17.6	35.1
$d_{c,DLS}$ **[mm]**	47.0		48.0	
$d_{c,ULS}$ **[mm]**	49.0		48.0	
max possible a_g for the ULS [g]	0.21		0.17	

Pushover curves corresponding to the results presented in Table 1 are shown in Fig. 3, in which graphs are organized so that the results obtained for different levels of seismic hazard can be compared directly. Target displacements and capacities provided in Table 1 are also marked. From the results presented in Fig. 3, several important observations can be made, as discussed below.
As expected, in the case of the upgraded tower somewhat larger base shear force can be observed than in the case of the existing one, resulting from the mass increase. Also, the upgraded tower has a slightly smaller stiffness than the existing one, due to the increased height.

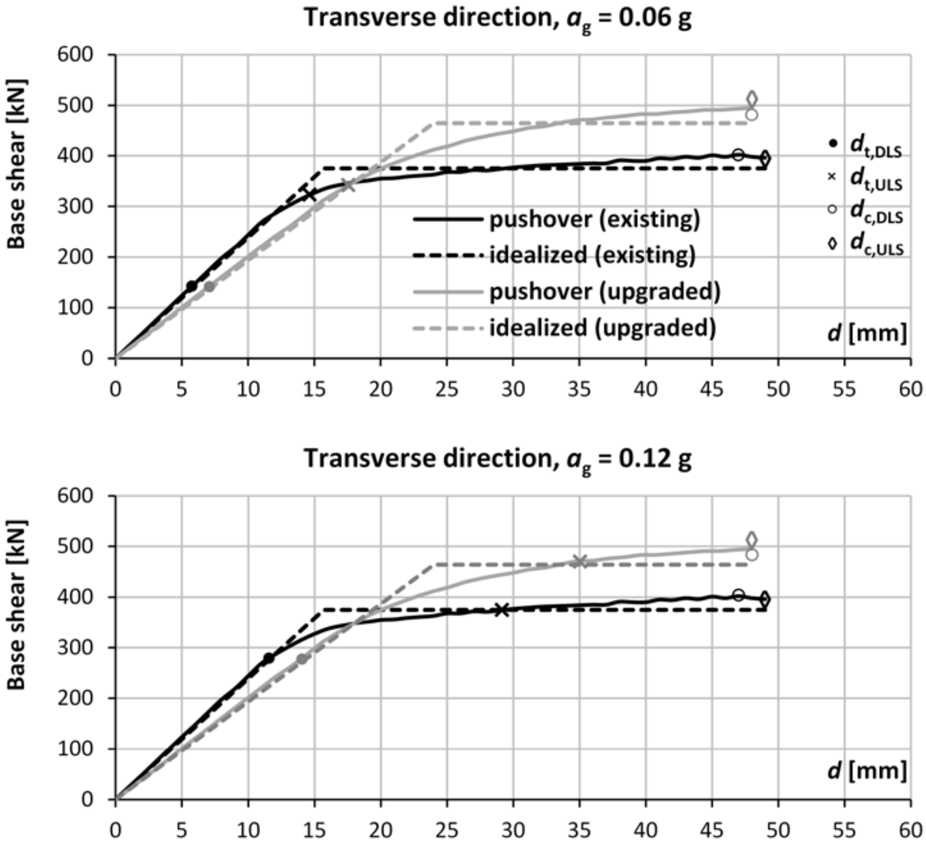

Figure 3 – Pushover curves with bilinear idealization, and demands and capacities obtained for the existing and upgraded towers

For both considered seismic hazard levels, displacement demands related to the DLS ($d_{t,DLS}$) are always on the parts of the pushover curves corresponding to the linear elastic structural response, which is an expected result.

In the case of a_g = 0.06 g the $d_{t,ULS}$ value obtained for the upgraded tower roughly corresponds to the linear elastic response, whereas in the case of the existing tower it slightly falls onto part of the pushover curve corresponding to the nonlinear response. It is obvious that for a_g = 0.06 g the DLS and ULS capacities of both towers are significantly higher than the demands. The values of $d_{t,ULS}$ obtained for a_g = 0.12 g for both considered towers fall onto the parts of the presented pushover curves related to the nonlinear response, and they significantly approach the values of capacities $d_{c,ULS}$. For both seismic hazard levels, in the case of the upgraded tower it can be seen that $d_{c,DLS} = d_{c,ULS}$, and that in the case of the existing one $d_{c,DLS}$ and $d_{c,ULS}$ values are pretty close. Moreover, the ULS capacities of the existing and upgraded towers are almost equal.

When it comes to the maximum possible a_g values corresponding to the ULS, it can be seen that the existing tower, as expected, has a higher capacity than the upgraded one, and that it has about 75% reserve in the maximum a_g value comparing to the design value of 0.12 g. On the other hand, the reserve in the maximum a_g approximately equal to 40% is present in the case of the upgraded tower, resulting from the increase in height.

5. CONCLUSIONS

A typical masonry church tower was analysed by pushover analysis performed on a spatial finite element model consisting of macro-elements, by considering different levels of seismic hazard (a_g = 0.06 g and a_g = 0.12 g). Examination of the differences in responses of the existing and upgraded towers structures led to several important observations:

- Displacement demands related to the DLS for both tower structures and both levels of seismic hazard correspond to the linear elastic structural response;
- For a_g = 0.06 g the displacement demand for the ULS obtained for the upgraded tower corresponds to the linear elastic structural response, whereas in the case of the existing tower it corresponds to the nonlinear response;
- The DLS and ULS displacement capacities of both tower structures are significantly higher than the demands when a_g = 0.06 g;
- When a_g = 0.12 g, the displacement demands for the ULS correspond to the nonlinear response for both considered towers, and they significantly approach to the corresponding capacities;
- For both seismic hazard levels the DLS and ULS capacities of both towers taken into consideration is almost equal;
- In terms of the maximum possible values of a_g, reserves of 75% and 40% were observed for the existing and upgraded towers, respectively.

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ODGOVOR POSTOJEĆEG I NADOGRAĐENOG ZIDANOG TORNJA CRKVE UZIMAJUĆI U OBZIR PROMENU SEIZMIČKOG HAZARDA

Резиме: *Tipičan zidani toranj crkve je analiziran sprovođenjem pushover analize na prostornom modelu formiranom od makro-elemenata. Za dva različita nivoa seizmičkog hazarda, ispitane su i diskutovane razlike u odgovorima postojeće i nadograđene konstrukcije tornja. Posebna pažnja je posvećena analizi zahteva po pomeranjima i kapacitetima koji odgovaraju stanjima ograničenih oštećenja i nosivosti, prema odredbama Evrokoda 8. Dodatno, određene su maksimalne moguće vrednosti projektnog ubrzanja tla za razmatrane slučajeve u cilju određivanja rezerve u kapacitetima konstrukcija.*

Кључне речи: *zidani toranj crkve, seizmički hazard, pushover analiza, Evrokod*