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# Seismic retrofit of an existing grammar school masonry building

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## Abstract

Application of modern seismic design codes in everyday practice imposes a need for the seismic retrofit of existing structures, which represents a challenging task when it comes to old masonry buildings. Many of such buildings were built long before design codes, not just seismic ones, and in many cases their structural performances do not conform with the current design criteria. Seismic response of existing masonry buildings, especially public ones, has a strong impact on the society, and therefore deserves a proper attention. There is a constant need for an improvement of the seismic capacity of old masonry building structures. One of such examples is presented in this paper, which deals with the seismic retrofit of an existing grammar school masonry building in Novi Sad, Serbia. The building was built about eighty years ago, before any design codes. It consists of a basement, ground floor and two upper floors. Structural walls were constructed by using solid bricks without reinforcement, and ribbed floor slabs were made out of reinforced concrete. During the on-site inspections no vertical confining elements were observed, and only some horizontal confining elements were found at floor levels. What makes the retrofit project special is the fact that a new storey is planned to be built upon the existing top storey in order to expand the school's capacity, which has further complicated the analysis and design. This paper presents the most important properties of the considered masonry building, the results of nonlinear static (pushover) analyses performed on a spatial mathematical model of the structure with the additional storey, and the applied retrofit solutions used to achieve the necessary seismic capacity with the respect to the provisions of Eurocode 8.

***Key words:*** masonry building, seismic capacity, retrofit, pushover analysis, Eurocode 8

# 1 Introduction

A proper assessment of the response of an existing masonry building during an earthquake represents a complex task, particularly in cases of existing structures. In general, a high level of knowledge on all relevant material and structural properties is required since any significant lack of data, combined with all uncertainties related to the seismic action, can lead to unreliable results. Also, structural modelling often has a crucial impact on the output. By taking into account the fact that existing masonry buildings are quite common, it is obvious that their seismic assessment needs to be conducted with special caution. When a retrofit of an existing masonry building is needed, the task becomes even more complex.

Modern design codes for the design and analysis of masonry buildings cover a fair range of common buildings in practice, and the application of Eurocode 6 (Part 1-1 [1]) coupled with Eurocode 8 (Part 1 [2] and Part 3 [3]) generally leads to appropriate structural solutions. In order to properly assess the behaviour of an existing masonry building exposed to the seismic action, at least the nonlinear static (pushover) analysis needs to be performed, which is in accordance with Eurocode 8. However, not enough structural modelling guidelines can be found in design codes, which represents a problem in certain cases. Recently, several useful software tools have been developed on the basis of the finite element method, experimentally obtained data, and observations on earthquake-damaged structures, which implies that the most commonly used "hand" calculations and checks shall be replaced with more sophisticated approaches.

This paper deals with the seismic retrofit of an existing grammar school masonry building in Novi Sad, Serbia, and presents the most important building properties, results of the nonlinear static (pushover) analysis performed on a spatial mathematical model of the structure with an additional storey, and the applied retrofit solutions used to achieve the necessary seismic capacity with the respect to the provisions of Eurocode 8.

# 2 Building description and applied loads and actions

## 2.1 Existing building

The existing grammar school masonry building was built about eighty years ago, before any design codes. It consists of a basement, ground floor and two upper floors, as shown in Fig. 1(a, b). Structural walls were constructed by using solid bricks without reinforcement, and ribbed floor slabs were made out of reinforced concrete (RC). During the on-site inspections no vertical confining elements were observed, and only some horizontal confining elements were found at floor levels. However, RC columns between windows in the longitudinal façades were observed (five in total).

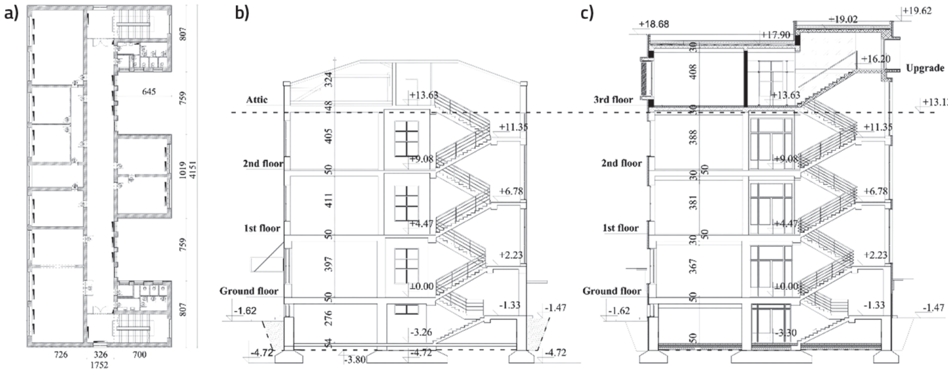



**Figure 1. a) Characteristic storey layout, and transverse cross-sections of the b) existing and c) upgraded building**

The existing structure is quite old, and in the available documentation no data was provided on the properties of the applied materials. Consequently, the material properties used in the analysis were adopted based on experience and relevant literature (e.g. recommendations from [4] were partially taken into account).

Compressive strengths of solid bricks ($f_b$) and mortar ($f_m$) recommended in [4] were used, and the characteristic compressive strength of masonry ($f_k$) was calculated based on [1] as 2.76 MPa. Conservatively, it was adopted that $f_k$ = 2.5 MPa. Characteristic initial shear strength of masonry under zero compressive stress ($f_{vko}$) was adopted as 0.10 MPa, whereas for the limit value of characteristic masonry shear strength $f_{vlt}$ = 0.65 MPa was used. Furthermore, short term secant modulus of elasticity of masonry ($E$) was determined according to [1] as $E = 1000f_k$ = 2500 MPa. For shear modulus ($G$) of masonry, a conservative value of 375 GPa was adopted, according to recommendations provided in [5]. In the case of RC columns, beams and lintels, concrete C12/15 was adopted according to [6], along with the smooth reinforcing bars with yield strength $f_y$ = 240 MPa. Volume weights of masonry (with mortar) and RC were taken as 19.0 and 25.0 kN/m$^3$, respectively.

Depending on their position, the thickness of basement walls is 25, 51 and 64 cm, the thickness of ground floor walls is 25, 38 and 51 cm, and in the above storeys the wall thickness is 25 and 38 cm. The above mentioned five RC columns in façades are 64 cm wide, whereas their thickness is 51 cm in the basement and 38 cm elsewhere. The reinforcement of these columns was not determined during the on-site inspections.

Ribbed one-way RC floor slabs are 43 cm thick, with ribs placed at the distance of 80 cm, and with a 5 cm thick concrete slab connecting them. The rib cross-section is variable, forming a vault in the non-bearing slab direction. Each rib is reinforced with two 16 mm diameter bars, and with 6 mm diameter stirrups spaced at 30 cm.

### 2.2 Upgraded building

What makes the retrofit project special is the fact that a new storey is planned to be built upon the existing top storey in order to expand the school's capacity, which has further complicated the analysis and design. The additional storey can be seen in the cross-section shown in Fig. 1.c. According to the original design of the additional storey, aerated concrete 25 and 38 cm thick blocks (depending on their position) are planned to be used for the wall construction, along with the horizontal and vertical confining RC elements with appropriate dimensions. A flat roof structure is designed as a solid 18 cm thick RC slab.
When it comes to material properties, according to [7] and [8], the following properties were adopted for the aerated concrete blocks: $f_k$ = 1.74 MPa, $f_{vk0}$ = 0.30 MPa, $f_{vlt}$ = 0.16 MPa i $E$ = 2500 MPa. Recommendations provided in [5] were used for the estimation of $G$, which was taken to be equal to 500 MPa. For the RC elements, concrete C25/30 ([6]) was adopted, along with the ribbed reinforcement B500B with $f_y$ = 500 MPa. Volume weights of masonry (with mortar) and RC were taken as 8.0 and 25.0 kN/m$^3$, respectively. Vertical confining elements were reinforced with four 14 mm diameter longitudinal bars and 8 mm diameter stirrups spaced at 15 cm. Horizontal confining elements were reinforced in the same way as the vertical ones. RC columns, which were designed at the positions of the existing façades columns, were reinforced with six 14 mm diameter longitudinal bars and 8 mm diameter stirrups spaced at 15 cm. The reinforcement of the roof slab is not relevant for this paper, and is therefore omitted.

### 2.3 Applied loads and actions

Besides self-weight of the building, the additional dead load applied on the first and second storey slabs amounted to 1.5 kN/m$^2$, whereas on the third storey and roof slabs it amounted to 2.5 kN/m$^2$. A variable load on every slab (except the roof) amounted to 2.5 kN/m$^2$, and the snow load considered on the roof slab amounted to 1.0 kN/m$^2$ (which only influenced static analysis, not discussed in this paper). Eurocode 8 type 1 elastic spectrum (corresponding to 5 % damping) for soil type C and $a_{gR}$ = 0.10 g represented the seismic input, with the respect to the building importance class III. The verifications of the ultimate limit state (ULS) and damage limitation state (DLS) were performed by considering all relevant Eurocode 8 provisions.

## 3 Structural modelling and analysis parameters

When the finite element method is applied on masonry buildings, macro-element models are commonly used. For the purpose of the project presented in this paper, the equivalent frame approach was used, which enabled modelling of all additional elements, such as beams, columns and confining elements, along with their nonlinear behaviour. More relevant details on the equivalent frame models can be found elsewhere (see e.g. [9-12]).

In the pushover analysis, the characteristic values of material properties were used, although Eurocode 8 suggests that the mean values should be applied. This way, a slight amount of conservatism was introduced. 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. All floor slabs were modelled as rigid diaphragms in their own planes. All openings in walls, along with parapets and spandrels, were modelled as realistically as possible.
The capacity of individual elements, in terms of drift limits, was taken into account according to Part 3 of Eurocode 8 ([3]). In the case of the ULS, which is roughly equivalent to the limit state of significant damage (SD) defined in [3], the drift limits taken into account amounted to 0.80 and 0.40 % for flexure and shear, respectively. In the case of the DLS, which was found to be the irrelevant in the considered case, the assessment was conducted with the respect to the inter-storey drift limit, as given in the Part 1 of Eurocode 8 ([2]).
Pushover analysis was performed by using the modal and uniform distributions of the seismic force, in both principal (X and Y) positive and negative directions. The accidental torsional effects were taken into account through the application of 5 % load eccentricity. The eccentricity was considered only in one (positive) direction. Three-dimensional representation of mathematical models of the upgraded building structure (without the basement) before and after the retrofit are shown in Fig. 2.

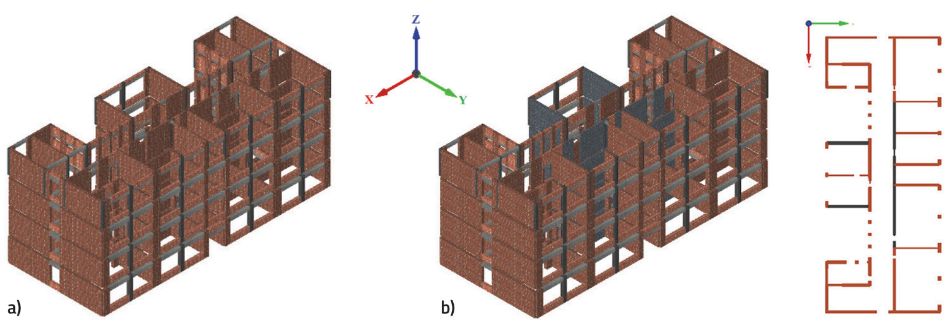



**Figure 2. Mathematical models of the considered upgraded building: a) before; b) after the retrofit**

## 4 Results

In order to determine target displacements, the N2 method was used, as incorporated in Eurocode 8 (for more details about the method see [13] and [14]). The obtained target displacements were compared with the corresponding capacities. First, the results obtained for the non-retrofitted building structure are discussed. Afterwards, the retrofit solutions are described, and then the results for the retrofitted building are presented. For convenience, the results for the non-retrofitted and retrofitted structure are shown in the same graphs.

## 4.1 Non-retrofitted structure

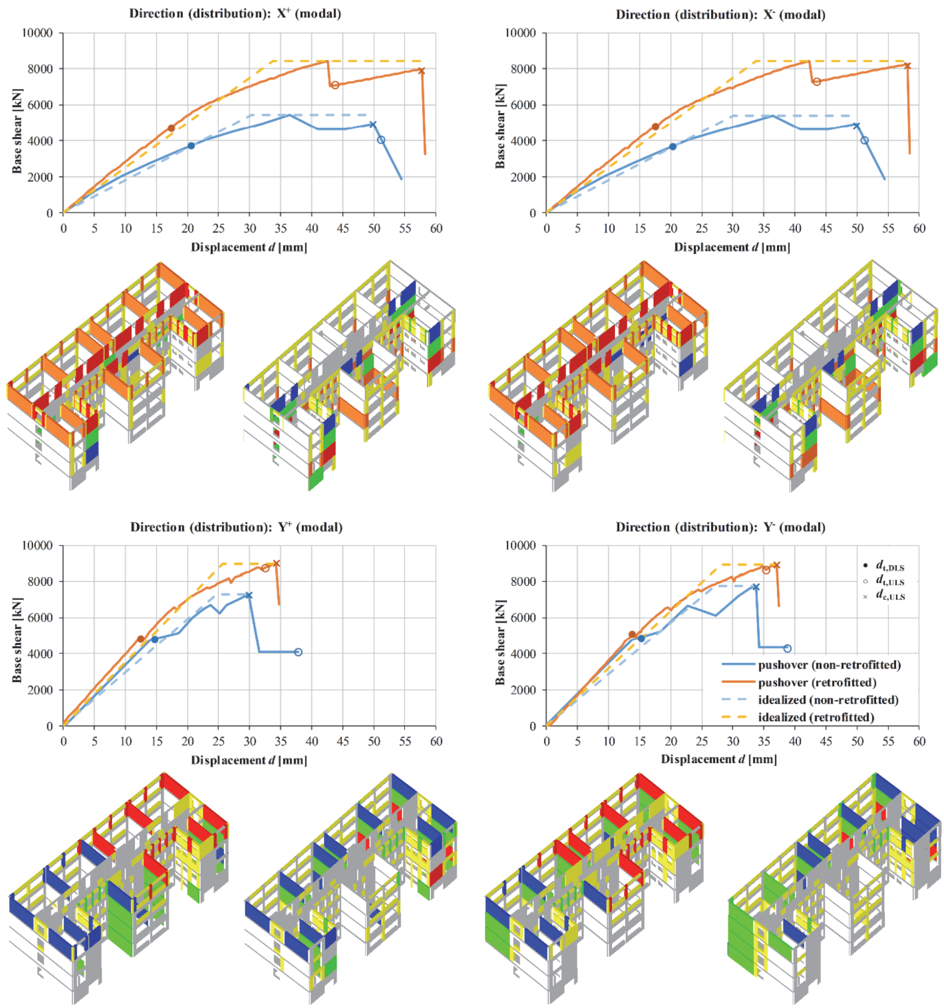
Pushover curves obtained for the modal and uniform seismic force distributions are shown in Figs. 3 and 4, respectively (below each graph, the left figure corresponds to the non-retrofitted structure). It can be seen that in all considered cases the DLS demands were relatively low, which confirms the above statement that the DLS was found to be irrelevant for the considered building. Thus, it will not be discussed further. On the other hand, the seismic capacity of the structure for the ULS was in all cases smaller than the demand. When it comes to the X direction, the capacities were closer to the demands in the case of the modal distribution, whereas in the Y direction, they were closer in the case of the uniform distribution. Obviously, the retrofit of the building is necessary.

## 4.2 Retrofit solutions

Two retrofit solutions were applied. The first one was applied to the existing third storey slab, which needed to be strengthened in order to carry the additional loads. This was done by adding two 14 mm diameter bars in the bottom zone of each rib, and by casting a 7 cm thick concrete topping (with Q-221 reinforcing mesh) on top of the existing 5 cm slab. In order to ensure the shear capacity of the connection between the existing slab and new concrete topping, dowels with 8 mm diameter and 10 cm length were used, and they were placed above the ribs. Thus, the existing third storey slab (which had noticeable cracks in its top zone) was strengthened, and the proper transfer of horizontal forces was ensured. The second retrofit solution was applied to the existing masonry walls, marked with grey colour in Fig. 2.b. A centre part of the longitudinal wall in the middle of the building and two transverse walls in the inner axes were strengthened with reinforcement, by adding Q-335 meshes along their whole height on both faces (starting from the foundation), and by using a 3 cm thick shotcrete C25/30. Also, aerated concrete walls in the upgraded storey that are above the retrofitted existing walls were reinforced in the same manner. Several characteristic retrofit details are shown in Fig. 5 (chapter 4.3).

## 4.3 Retrofitted structure

Pushover curves corresponding to the retrofitted structure, obtained for the modal and uniform seismic force distributions, are also shown in Figs. 3 and 4, respectively (below each graph, the right figure corresponds to the retrofitted structure). It can be seen that in all considered cases the ULS capacities were larger than the demands, and that the most critical case was the $Y^-$ for the uniform force distribution. Obviously, the chosen retrofit solution for walls led to the satisfactory results.




*Graph legend:*

- **$d_{t,DLS}$** ○ **$d_{t,ULS}$** - Target displacements for the DLS and ULS (respectively)

× **$d_{c,ULS}$** - Capacity corresponding to the force reduction on the capacity curve prior to exceedance equal to or more than 20% (obtained for the MDOF system)

*Damage status* shows how the element forces are related to ultimate capacities [12]:

**SD** - total shear capacity exhausted, full shear failure
**FD** - total flexural capacity exhausted, full flexural failure
**MV** - both moment and shear ultimate forces are reached, plastic behaviour in flexure and shear
**M** - moment capacity reached, plastic behaviour in bending/flexure
**V** - shear capacity reached, plastic behaviour in shear


**Figure 3. Results of the pushover analysis for the non-retrofitted and retrofitted building and modal force distribution**

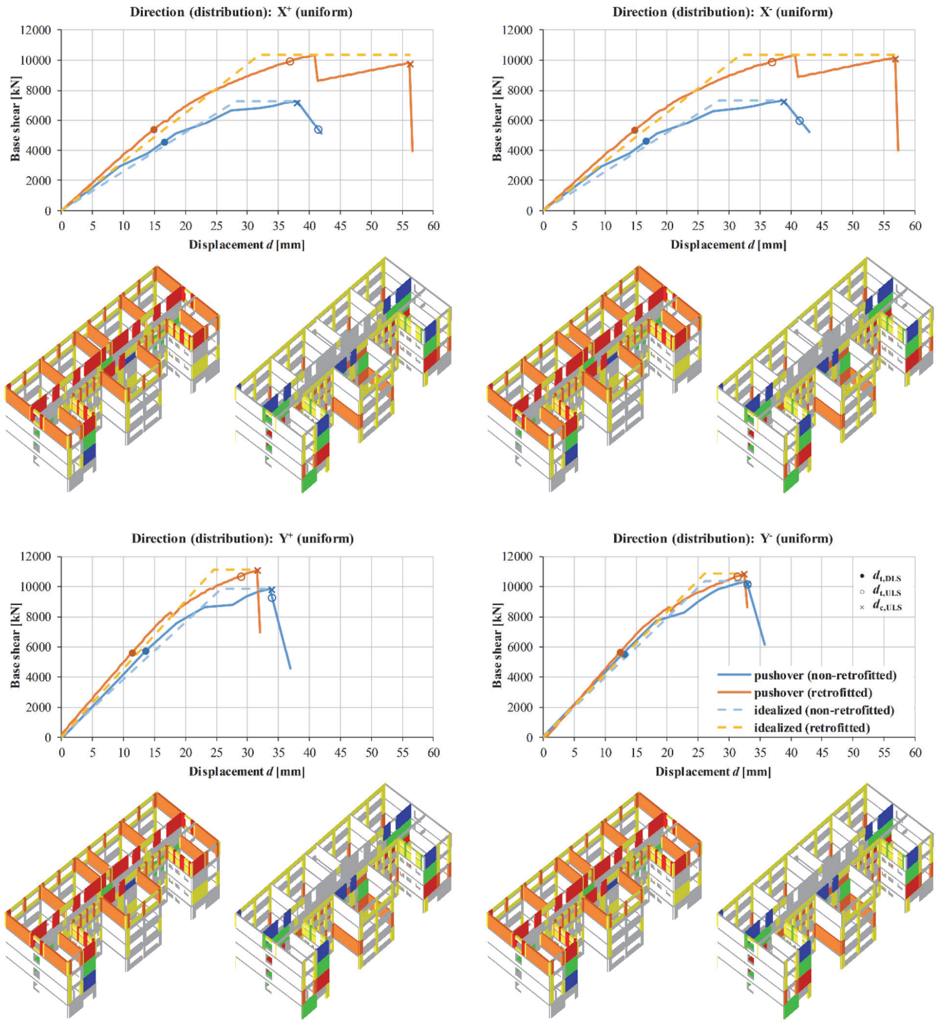



**Figure 4. Results of the pushover analysis for the non-retrofitted and retrofitted building and uniform force distribution**

Furthermore, a comparison of the pushover curves determined for the non-retrofitted and retrofitted structure clearly illustrates the retrofitting benefits. The adopted strengthening of walls turned out to be very practical. In terms of costs, the solution is rational, which confirms the outcomes of some previous studies (see e.g. [15]).

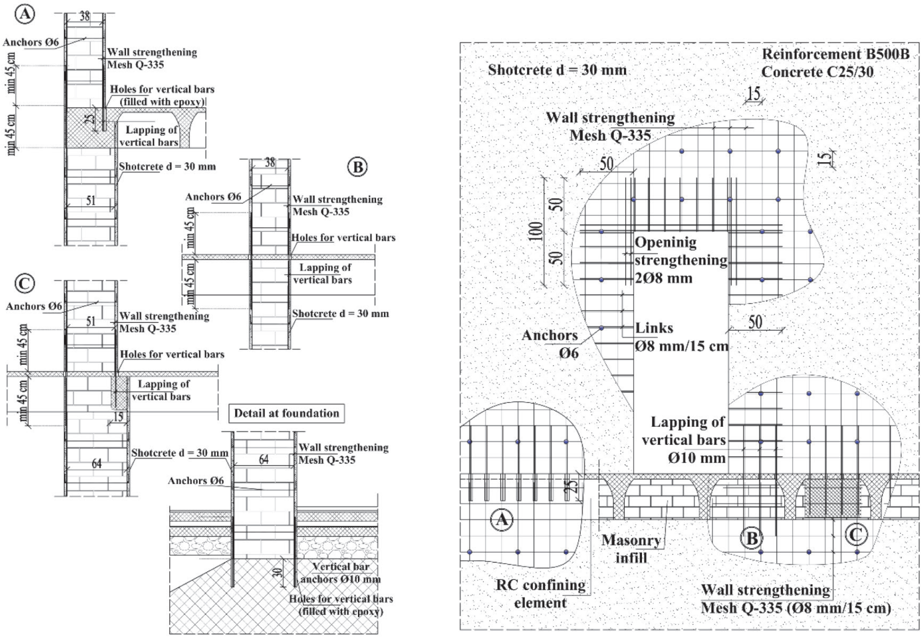



**Figure 5. Characteristic retrofit details**

# 5 Conclusions

This paper shows the seismic retrofit of an existing grammar school masonry building in Novi Sad, Serbia. What makes the retrofit project special is the fact that a new storey is planned to be built upon the existing top storey in order to expand the school's capacity, which has further complicated the analysis and design.

The most important building properties, results of the nonlinear static (pushover) analysis performed on a spatial mathematical model of the structure with an additional storey, and the applied retrofit solutions used to achieve the necessary seismic capacity according to the provisions of Eurocode 8, were presented in the paper.

For the considered building, it was observed that both the modal and uniform seismic force distributions can be relevant, so it is not possible to know in advance which one should be applied in the pushover analysis. Thus, the provisions of Eurocode 8 which explicitly state that at least two force distributions ought to be taken into account, should not be ignored in practice.

Finally, it was shown that strengthening of walls by adding reinforcing meshes and shotcrete on their both faces led to the sufficient increase of the seismic capacity. A comparison of the pushover curves obtained for the non-retrofitted and retrofitted structure clearly illustrated the retrofitting benefits. In the case of the considered building, the wall strengthening solution turned out to be quite practical, and rational in terms of costs.

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