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SEISMIC RETROFITTING OF SULTAN MURAT MOSQUE' CLOCK TOWER IN SKOPJE USING INNOVATIVE MATERIALS

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Abstract

The structure of the Clock Tower in Skopje is massive, constructed of stone masonry with a height of about 29 meters and brick masonry in the upper octagonal part. It was damaged during the Skopje Earthquake in 1963. A rehabilitation of the structure was carried out in 1964, but without a seismic strengthening. Recently, field investigations were carried out and a three-dimensional analysis of the structure in its actual state for gravitational and seismic forces was performed. It has been concluded that it is necessary to perform protective interventions to ensure longer-term protection, stability and further existence in the form of repair of existing structural damage, structural consolidation, repair of damaged timber elements and strengthening. To improve the behaviour and resistance to dynamic impacts, a technical solution for consolidation and retrofitting has been proposed. The basic concept consists of existing structural damage rehabilitation, structural consolidation with systemic injection, rehabilitation of load-bearing wooden beams and braces and strengthening the whole structure. Due to the specificity of the structure of the Clock Tower in terms of its shape, used material, structural system and the type of structural elements, as well as its importance as a cultural and historical heritage, which requires respect of certain conservation principles and rules, the options for rehabilitation and strengthening are strictly limited. It excludes inserting new structural elements that would provide sufficient ductility during maximum expected earthquakes. The rehabilitation and strengthening were carried out with innovative composite materials that were experimentally verified, but never applied before. The analysis results show that the retrofitted structure can withstand the maximum expected seismic loads according to the actual technical regulations.

Keywords: historic buildings, seismic retrofitting, innovative advanced composite materials, analytical investigation

1. Introduction

The Clock Tower building in Skopje is located in the Sultan Murat Mosque complex. According to information, it was constructed between 1566 and 1573 and was also the first clock tower on the territory of the Ottoman State [1]. It was built on the foundations of a former medieval defence tower. It was previously constructed as a wooden structure, but later it was completely constructed as a brick masonry. During the arson of the city of Skopje and the great fire from 1689, the tower suffered a great damage, but it remained in its original shape until 1904, when it was rearranged and got its present shape (Fig. 1).

During the 1963 Skopje Earthquake, the tower was damaged and the clock mechanism was lost. Later, the structure was repaired, but without its clock mechanism. The new clocks on the tower were added on May 26, 2008, as part of the Chair Municipality project for its reconstruction.

In the period after the earthquake, a report for structural strengthening was prepared. The strengthening was planned to be performed inserting reinforced concrete vertical and horizontal belt courses [2]. From the recent on-site inspection of the building, it was ascertained that the project was not implemented. Damage and cracks of the structural walls were still visible on the cleaned facade masonry as well as in the interior (Fig. 2).

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Figure 1. The Sultan Murat Mosque Clock Tower (postcard from 1909) (left); Appearance in 2018 (right)

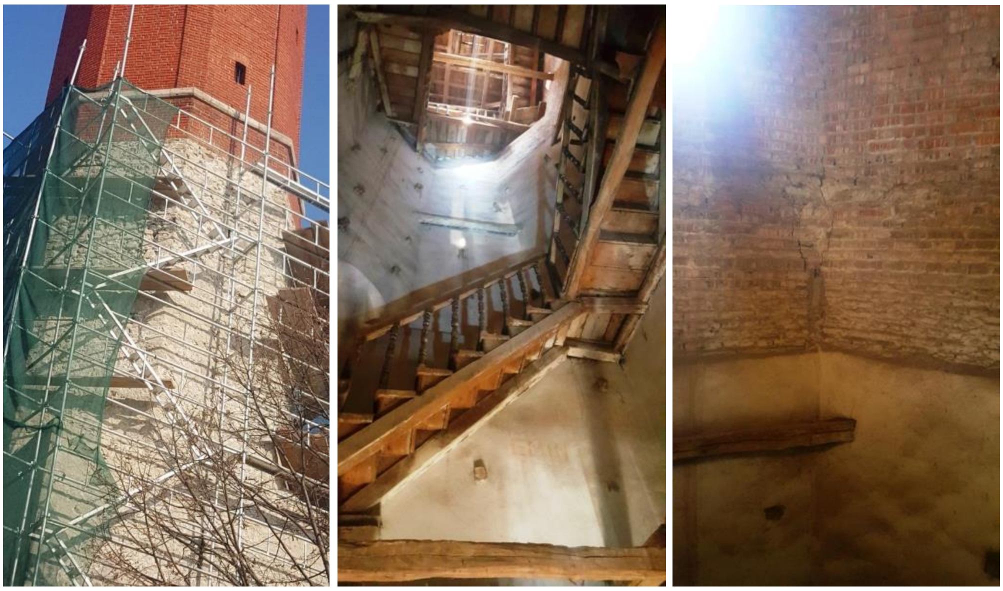

Figure 2. The condition of the structural masonry walls before rehabilitation

2. Structural concept and structural damages

From a structural point of view, the tower is constructed as massive stone masonry in lime mortar up to elevation +15.00 m, and from elevation +15.00 to the top (elevation 29.00 m) as brick masonry in lime mortar. In the lower part of the stone masonry (up to elevation +8.50), which is probably a remnant of the medieval defensive tower, the shape in plan is in the form of a square with a side length of 5.50 m and a wall thickness of 1.0 m, and then it turns into an octagonal shape. From elevation +8.50 m up to elevation +24.30 m the external dimensions are constant, and above this level to the top of the building there are two cascading reductions in size and in wall thickness to 80 cm and 60 cm. The connections between these various levels are constructively resolved by supporting load-bearing multi-layered wooden beams and with no direct contact between the lower and the upper masonry.

From the inspection of the building, it was generally concluded that the masonry was in good condition with some visible damages in the form of diagonal and vertical cracks on the facade stone

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walls, on the inner side of the brick walls, damages from a deterioration of the wooden beams and the mortar due to aging, as well as damages due to an insufficiently regular maintenance.

For the structural analysis purposes, additional research was carried out to determine the foundation depth. The tower is founded on a sloping terrain, so the southern side is at an elevation of +0.00 m, while the elevation of the northern part is at an elevation of -2.00 m. From the investigation probe along the northeast corner of the building, the identification of the foundation was carried out (Fig. 3). The foundation is constructed of a solid stone masonry in lime mortar without extension up to elevation -4.00 m and it is a continuation of the stone masonry above the ground level.

Figure 3. An exploratory borehole along the northeast corner of the tower

3. Structural analysis of the tower's state before the seismic retrofitting

The structural analysis of the tower was carried out for gravitational and seismic forces, whereby the building is treated in accordance with the actual technical regulations for the appropriate category of the building according to its purpose. A three-dimensional static and seismic analysis was performed applying the finite element method using the SAP2000 software package. A medium-dense network of elements was adopted, covering the global geometric characteristics without considering the inhomogeneity of the built-in material (stone, brick and mortar). For the purposes of this analysis, a mathematical model for the structural system of the building was prepared (Fig. 4), where a three-dimensional finite element SOLID with eight nodes was used to model the load-bearing massive walls.

For the structure modelled in this way, a modal analysis was first performed, then an analysis with applied gravitational vertical loads and equivalent seismic forces, obtained according to the valid national regulations [3]. The following values, (typical for Ottoman monuments in N. Macedonia) are adopted for the input physical-mechanical parameters for stone masonry and brick masonry:

Weight of the stone masonry, W_s = 20 kN/m^3
Elasticity modulus of the stone masonry, E_s= 1100 MPa
Weight of the brick masonry, W_b = 17 kN/m^3
Elasticity modulus of the brick masonry, E_b = 850 MPa

According to the national regulations for high-rise buildings in seismically active regions [3], as well as considering that the tower is a cultural and historical heritage, the seismic analysis had been conducted as for a first category building, with the seismic force intensity equal to 30% of the total weight of the building.

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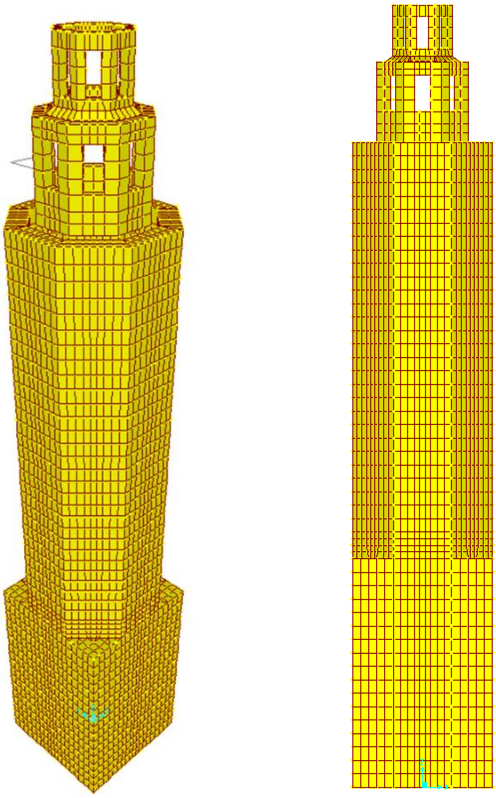

Figure 4. 3D solid finite element model of the tower

The total seismic coefficient was obtained by the Eq. 1:

$$K = K_0 * K_s * K_d * K_p \tag{1}$$

is K=0.3, where the values of each individual coefficients are given below:

Building category I coeff. (Ko = 1.5)
Site seismicity MCS IX coeff. (Ks =0.1)
Soil class I coeff. (Kd=1.0)
Ductility and damping coeff. for plane masonry (Kp= 2.0)

Since there is no direct connection of the masonry from elevation +24.30 above, further analyses were performed up to that level by considering the influence of the upper part of the building only as an additional weight. According to the numerical analysis the natural periods on both orthogonal directions have the same value ($T_{x\text{-}x}$ = $T_{y\text{-}y}$ = 0.81s), while the natural period in torsion is lower (T_{tor}=0.21s). These values are within the expected limits for natural period of clock towers, The total displacements from seismic force in the longitudinal and transverse directions are 11.2 cm in both directions (Fig. 5) and they are higher than the maximum allowed according to regulations (δ_{max}=H/600=2900/600=4.83cm).

The resulting stress-deformation state indicates a sufficient capacity of stability and reliability for gravitational loads since the maximum compressive stresses are lower than the expected ultimate compressive strength of stone masonry. However, considering the maximum expected seismic impacts, tensile stresses higher than the expected tensile strength of the masonry occur in local zones, which would result in structural damage due to the expected seismic action (Fig. 6). Thus, the need for structural consolidation and strengthening was determined in order the seismic protection level to be achieved.

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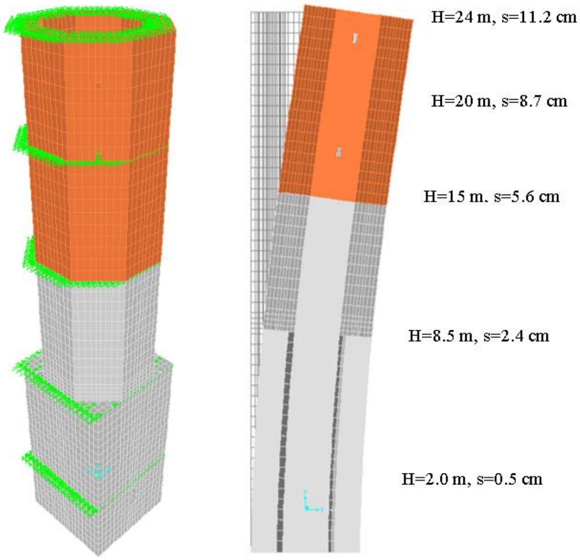

Figure 5. Unretrofitted state - elevations of applied seismic forces (left), displacements due to the design seismic actions (right)

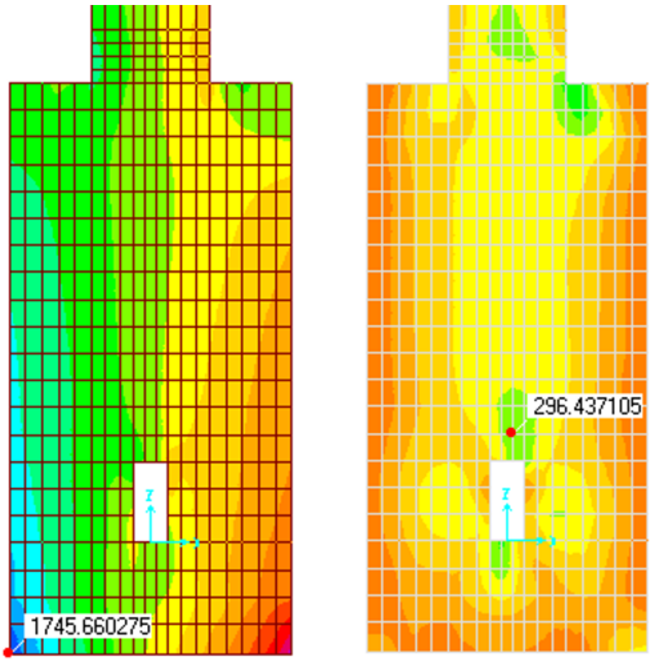

Figure 6. Unretrofitted state - stress in kPa under gravity loads and seismic actions in critical points – vertical tensile stress (left), shear stress (right)

An analysis of the overturning moments for the maximum expected seismic forces was also performed. The self-weight combined with the passive earth pressure provides a stabilizing moment of 26,807 kNm, while the seismic forces cause an overturning moment of 34,966 kNm. Therefore, it was concluded that it is necessary to strengthen (expand) the foundation to obtain a higher safety coefficient against overturning.

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4. Technical solution for seismic retrofitting

The measures for improving the seismic reliability of culturally historical buildings, are defined with specific conditions that depend on the historical, architectural and cultural value of the building, on the seismic hazard level, but also on the possibility of applying an appropriate rehabilitation and strengthening measures. Buildings such as the Clock Tower in Skopje, due to their shape, built-in materials, structural system and especially the type of structural elements, as well as to respect certain conservation principles and rules, do not allow inserting enough new structural elements that would provide sufficient ductility during expected earthquakes.

To improve the behaviour and resistance to dynamic actions, the technical solution had been proposed [4], the basic concept consists of:

- Repair and rehabilitation of existing structural damage;
- Structural consolidation with systemic injection;
- Rehabilitation of load-bearing wooden beams and braces with modern composite materials;
- Strengthening the structure with advanced composite materials.

4.1 Rehabilitation, structural consolidation and strengthening

This measure included repair of visible cracks, as well as all existing cracks that were noticed after the surface mortar layer had been removed.

The process of repairing structural cracks started with cleaning the facade mortar and injection of mixtures based on lime mortar with additives, (crushed brick, pozzolan), fluidity improvement additives, improvement strength and adhesion with the substrate.

For structural consolidation and overall increase of the masonry compactness and thereby improving the load-bearing capacity and integrity of the masonry, a systemic injection had been performed. The process included cleaning the inner surface of the elements and removing the damaged and labile parts. The injection was carried out with similar mixtures with increased fluidity, under a pressure of 1-1.2 atmospheres with specialized equipment, along previously systemically laid pipes and in the direction from bottom to top.

Considering the significant role of the wooden beams at the level +24.30 and +27.00 as basic load-bearing elements for the masonry above them, as well as the role of the wooden braces as elements that receive the tension stresses that the masonry has no capacity to receive, the function of all wooden elements had been checked. All damaged or worn-out wooden elements were repaired or replaced with new ones. For effective rehabilitation and strengthening of load-bearing wooden beams, they were cover with polymer cloth or a belt reinforced with carbon fiber (CFRP Wrap, CFRP Laminate) after the preliminary cleaning and treatment of the surface layers.

The basic concept of the strengthening consisted of horizontal and vertical belts of modern polymer cloth, reinforced with fibers in a layer of lime mortar, which should ensure complete coverage of the tensile stress zones. Apart from absorbing the tension forces and preventing damage to the masonry, this system also enables greater integrity and increases the stiffness and ductility of the structure during dynamic loads.

For the implementation of the strengthening system, a modern, previously experimentally verified system (ROFIX SismaCalce [5]) has been adopted, which consists of three layers: (1) a primary layer of basic hydraulic lime mortar, (2) a layer of a mesh of polymer-coated aramid and glass fibers (Fig. 7) and (3) a layer of facade lime mortar of at least 2.0 cm.

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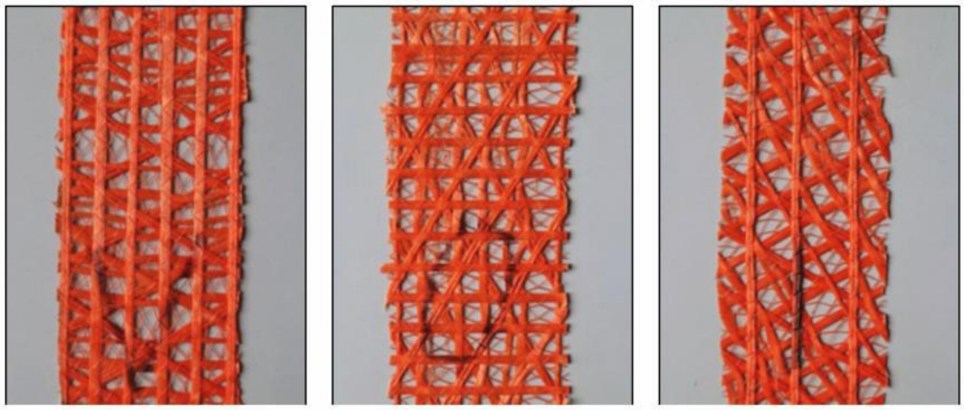

Figure 7. Used mesh of polymer-coated aramid and glass fibers

According to the conducted analyses and the possibilities for the application of the proposed system, the following strengthening elements were foreseen (Fig. 8):

- Vertical external elements VN1, VN2, VN3 and VN4 at the corners of the lower part of the stone masonry in the form of a square with a length of at least 320 cm and a height of at least 12.5 m (from elevation level -3.00 m to +8.50 m, plus overlapping length);
- Vertical external elements VN5, VN6, VN7 and VN8 on four of the eight sides of the octagonal part of stone masonry with a length of 220 cm and a height of 7.5 m (from elevation +8.50 m to +15.00, plus overlapping length);
- Vertical internal elements VV1, VV2, VV3 and VV4 on four of the eight sides of the brick masonry with a minimum length of 160 cm and a height of 10 m (from elevation +15.00 m to elevation +24.30 m, plus overlapping length). These elements were constructed on the inside of the walls due to the requirements for keeping the facade unchanged. In order to fully achieve the integrity effect, it is planned to connect these elements with composite bars (CFRP bars) placed next to the pilasters on the facade brick wall;
- Horizontal external elements HN1 and HN2 on the part of stone masonry in a square shape that go around the eternal facade on elevation +3.90 m and +8.00 m respectively, with a minimum width of 220 cm and 125 cm respectively and a length of 23 m;
- Horizontal external elements HN3 and HN4 on the section of stone masonry in an octagonal shape on elevation +10.50 m and +15.00 m respectively, with a width of at least 125 cm and a length of 18 m;
- Horizontal internal elements HV1, HV2 and HV3 on the section of brick masonry on elevation +16.90 m, +20.60 m and +23.70 m with widths of 3.20 m, 1.25 m and 1.25 m respectively, and a length of 13 m. The connection to the composite bars at the root of the pilasters described above applies to the connection to these elements as well. The connection should be achieved applying chromed steel stirrups inserted through the joints in the masonry on every 40 cm from elevation +16.40 m to elevation +23.40 m.

To consistently implement the strengthening below the ground level, based on the observation of the open probe, the knowledge obtained about the foundation and the calculations performed, a solution is provided for strengthening the foundation walls from elevation -4.00 m at least to elevation -2.00 m. It consists of a reinforced concrete wall, 25 cm thick with extra 80 cm of extension on the footing along the whole outer perimeter of the foundation walls after previous rehabilitation by injection and appropriate treatment of the surfaces. To achieve interaction with the existing foundation structure, chrome steel anchors with appropriate length were installed in previously drilled holes, filled with epoxy mortar. With such a solution, the reliability coefficient against overturning from maximum seismic forces have been significantly increased.

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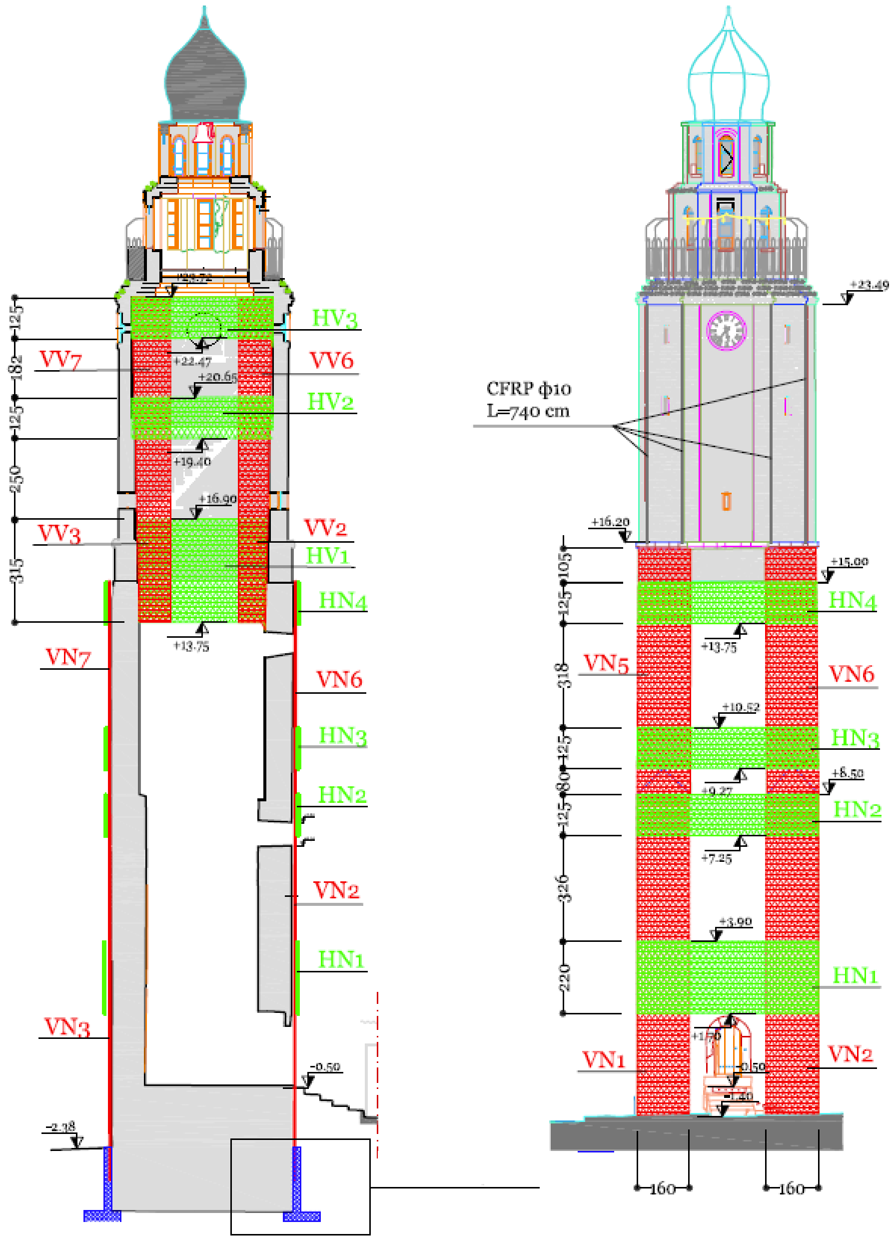

Figure 8. Strengthening elements – cross section (left), on the facade (right)

5. Structural analysis of the tower's state after the seismic retrofitting

For the strengthened state of the tower, a structural analysis for gravitational and seismic loads was carried out, whereby the structure is treated in accordance with the actual regulations and rules for the appropriate category of the building according to its meaning and purpose. For the purposes of this analysis, a mathematical model for the structural system was prepared (Fig. 9), where a two-dimensional finite element SHELL with four nodes was used to model the reinforcement elements.

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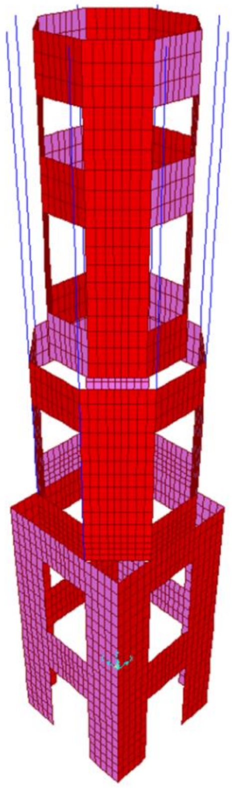

Figure 9. Numerical FEM model of the strengthening elements

For the strengthened structure modelled in this way, a modal analysis was performed, followed by an analysis from gravitational vertical loads as well as from equivalent seismic forces. According to the national regulations for high-rise buildings in seismically active regions [3], the seismic analysis had been conducted as for a first category building, with the seismic force intensity equal to 24% of the total weight of the building.

The mechanical parameters of the "ROFIX SismaCalce" system, were adopted according to the experimental verification results, obtained in IZIIS' laboratory [5]:

Weight W = 15.5 kN/m^3
Elasticity modulus (90^0) E^{90} = 51 000 MPa
Elasticity modulus (0^0) E^0 = 31 000 MPa

The total seismic coefficient was obtained using the Eq. 1 is K=0.24, where the values of each individual coefficients are given below:

Building category I coeff. (Ko = 1.5),
Site seismicity MCS IX coeff. (Ks =0.1),
Soil class I coeff. (Kd=1.0),
Ductility and damping coeff. for confined masonry (Kp= 1.6)

The natural periods of the retrofitted state $T_{x\text{-}x} = T_{y\text{-}y} = 0.51$s, T_{tor}=0.15s were obtained for the structure modelled in this way, which indicates a certain structural stiffening. The total displacements due to seismic force in the longitudinal and transverse direction are also reduced and within the allowed limits ($\delta_x = \delta_y = 4.8$ cm) (Fig. 10).

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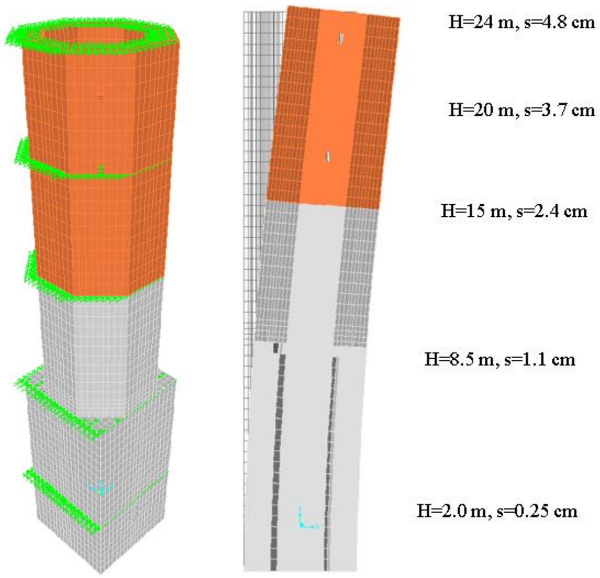

Figure 10. Retrofitted state - elevations of applied seismic forces (left), displacements due to the design seismic actions (right)

The resulting stress-deformation state of the retrofitted structure indicates that the maximum expected seismic impacts, tensile stresses is significantly reduced compared to the unretrofitted structure. Although remains higher than the expected tensile strength of the masonry in smaller local zones (Fig. 11), the global safety and stability of the structure to seismic events is significantly increased.

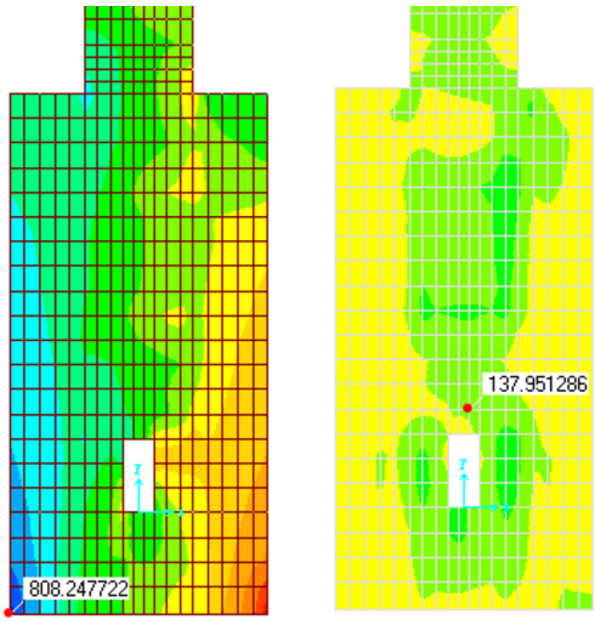

Figure 11. Retrofitted state - stress in kPa under gravity loads and seismic actions in critical points – vertical tensile stress (left), shear stress (right)

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6. Conclusions

The paper presents seismic retrofitting of an important historic building – the Clock Tower in Skopje using the innovative Rofix SismaCalce System, that was previously experimentally verified. This system consists of a primary layer of basic hydraulic lime mortar, a layer of a mesh of polymer-coated aramid and glass fibers and a final layer of facade lime mortar. Comparison of the results from analytical investigation for both unretrofitted and retrofitted structure shows that the global safety and stability of the retrofitted structure to seismic events is significantly improved.

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