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EXPERIMENTAL RESEARCH INTO THE RESPONSE OF HISTORICAL STRUCTURE TO THE EFFECTS OF NATURAL SEISMICITY

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ABSTRACT

The article presents the evaluation a building technical survey of a damaged historic rectory building in Luby near Cheb, it analyses the damage and proposes conceptual ideas for the rehabilitation design of a historic building damaged by the effects of natural seismicity applying the results and outputs of the NAKI II research project.

KEYWORDS

Natural seismicity; Historic building; Building technical survey; Stiffness of masonry structure; Additional prestressing

INTRODUCTION

The number of prominent historic and heritage buildings (over 40,000 immovable monuments situated on the territory of Bohemia, Moravia and Silesia), their cultural and historical value and the resources that must be spent on their restoration require special care for their protection and restoration.

The restoration and rehabilitation of historic and heritage masonry buildings, in many cases, requires the strengthening and rehabilitation of damaged and degraded masonry of pillars, walls and vaults of stone, brick, mixed or multi-leaf and half-timbered masonry, which is often in varying degrees of degradation and failure. In the case of historic buildings exposed to the effects of extraordinary loads and effects (undermined areas, areas susceptible to flooding and floodplains, mining areas, traffic-intensive areas, seismic areas), their spatial stability and residual structural capacity must be secured.

Severe earthquakes with a focus located on the territory of the Czech Republic or in its immediate vicinity are known from historical sources. Even today, weaker earthquakes are detected mainly in western and north-eastern Bohemia - in border areas with Germany and Poland. The strongest of them, which are also felt by the inhabitants, may be the cause of failure of historic buildings. According to [1], eight regions characterized by seismic hazards of up to the intensity of 7° on the MSK – 64 scale (ca 500 $mm.s^{-2}$) can be delimited in the Czech Republic – Kraslice Region (7°), Český Les (6°), the eastern part of the Krušné Mountains (6°), Silesia (6°), north-eastern Bohemia (7°), the Western Carpathians and the Carpathian Foredeep Basin (7°), South Bohemia (6°), South Moravia (6°). A total of over 18000 cultural monuments and 133 national cultural monuments are situated in these regions [2] (Figure 1).

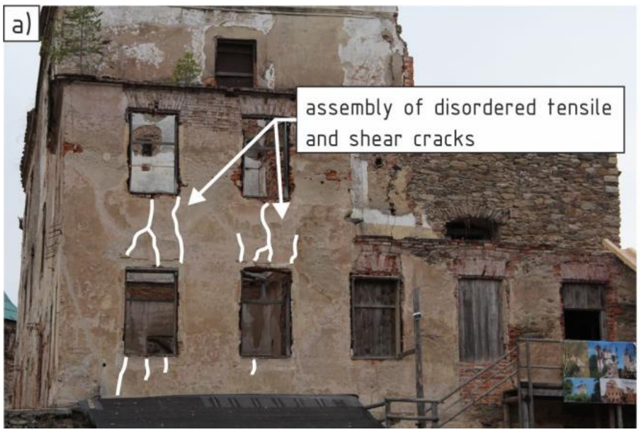

Fig. 1 - Illustration of damage to historic buildings in seismically active regions – a) Hartenberg Castle, Josefov (end of 12th and beginning of 13th century.), b) St. Anna's Church, Karlovy Vary – Sedlec (18th century)

The above facts testify to the need to address the issues related e.g. to the response of historic structures to the effects of natural seismicity, their potential consequences and the influence on their failure and residual structural capacity.

Earthquakes in the Czech Republic are usually concentrated in so-called swarms. An extremely strong swarm with the most active earthquakes was recorded in the Kraslice Region (western Bohemia) during the period of increased seismic activity at the turn of the 19th and 20th centuries. After a following period of rest, a stronger seismic swarm was registered in the area of the settlement of Nový Kostel in 1936 - 1937 and 1985 - 1986 reaching a local magnitude of 4.6 (intensity of 7° on MSK – 64). This earthquake intensity may cause damage to buildings - cracks, damage to plaster layers, falling off roofing, etc. More recently, increased seismic activity has been recorded again since 1997. At the end of 2000, a strong shock reached a local magnitude of 3.4 and was felt by the residents of Cheb, Sokolov, Karlovy Vary and Tachov Regions. A strong earthquake swarm was registered from August 2001, comprising over 1,500 earthquakes recorded in its eight phases, of which more than 5% were also felt by the local population [3]. The last major earthquake that has hit the Czech Republic so far occurred in May 2014. Its epicentre was located in the Cheb Region, in the settlement of Nový Kostel, 8.5 km below the Earth's surface, and the shocks could be felt as far as Central Bohemia and Prague. The earthquake's magnitude of 4.6 made it comparable to the earthquake of December 1985, and it became one of the strongest earthquakes in the last 100 years.

In addition to the above-mentioned areas, the majority of the territory of the Czech Republic is characterized by a seismic hazard of up to the fifth degree, and South Bohemia and Moravia up to the sixth degree MSK - 64 (the influence of Eastern Alpine and West Carpathian earthquakes). According to the seismic hazard map of the Czech Republic (Annex to the National Application Document to Eurocode 8), the manifestations of natural earthquakes with a macroseismic intensity in the range of 6 to 6.5 ° MSK - 64 can be expected mainly along the perimeter of the Bohemian Massif, with quasi-effective acceleration values ranging from 0.06 to 0.4 g.

Structures with insufficient spatial stiffness and insufficiently rigid foundation structures tend to be highly susceptible to dynamic effects caused by technical and natural seismicity [4]. The extreme sensitivity to dynamic effects caused by seismicity are, for example, typical of masonry buildings without bond beams, or beam and wall ties, with ineffective beam ties situated in degraded masonry, with an insufficiently rigid supporting system of vaults, buildings with yielding, e.g. joist and beam ceilings, with vaults without bowstrings and with insufficiently deep rigid foundations, unbonded foundations and inappropriate foundation subsoil [5, 6]. The type of masonry failure under dynamic loading with vibrations corresponds essentially to brittle fracture. At

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relatively low vibrations, the masonry breaks due to fatigue not only in joints, but also in masonry units. Vertical cracks occurring in the perimeter walls of castle and church towers are also, in many cases, caused by seismic effects (including low-cyclic temperature effects and dynamic effects of heavy bells) [6].

For the sake of simplicity, the movement during an earthquake can be assumed to be a simple harmonic motion - continuous, similar to the oscillatory motion characterised by an amplitude, period, velocity and acceleration. Building structures in regions with high seismicity are also assessed for the values caused by seismic movements of the foundation soil, for the effect of inertial forces acting at different points of the structure, concentrated at the level of individual floors.

The horizontal movement of the Earth's surface during an earthquake reaches 0.3 to 0.5 times (or more) the gravitational acceleration - this horizontal component has the most severe impacts on buildings. A frequent cause of failures of masonry structures is the relatively low tensile strength of masonry and low ductility, which is the cause of considerable sensitivity of these materials (masonry structures) to the effects of forced deformations. The consequences of this property are, in many cases, manifested locally by stress states with a significant tensile component. The severity and intensity of seismic (dynamic) effects spreading through the soil, caused primarily by natural seismicity, depends, among other things, on the execution method and the properties of the foundation structures of the building. The composition of the geological environment and its mechanical properties affect the magnitude of vibrations from the subsoil, which may be amplified or damped by this composition. Natural frequencies - of soils of the overlying formations on the bedrock - are a major agent in terms of the propagation of vibrations through the subsoil. In the conditions of the Czech Republic, the usual thickness of soils on the bedrock is 2-4 m. In this case, the natural frequencies of the soil on the bedrock may approach the natural frequencies of buildings, and, as a result, the transmission of vibrations from traffic into building structures is amplified by the so-called resonance effect [7]. The failure of masonry structures may also occur due to the secondary excitation of subsoil movements in the vicinity of non-stabilized geological conditions.

The effect of seismic undulation of the subsoil with direct contact is first transmitted to the foundation structure of the building, exerting cyclic horizontal deformations of the foundation in response to the cyclic movements of the subsoil, which are transmitted to higher floors via the underground (lowest) floor depending on the shear and flexural stiffness. The magnitude and type of the horizontal deformation depends mainly on the stiffness of the structure of individual floors or the substructure. The highest values of stresses or horizontal (shear) deformations related to the distribution of the load-bearing system's stiffness along the height of the building are on the lowest floors, situated between the foundations and the superstructure, or in the substructure. In this perspective, systems with a relatively low stiffness on these floors – e.g. spacious halls, temple naves, etc. - represent the weakest, critical point, usually with the lowest resistance to seismic effects [8, 9].

LUBY RECTORY

The in-situ experimental activity aimed at the influence of seismic effects on historic structures carried out in the historic building of the Luby Rectory included the building technical survey, numerical analysis and, based on them, the identification of the likely cause of failures in the building and design of potential rehabilitation measures.

Brief description of the building

The rectory building in the settlement of Luby was built at the end of the 17th and the beginning of the 18th century, and is situated in the immediate vicinity of St. Andrew's Church, which is dated back to the 11th century. In 1739, the rectory was reduced to ashes, and only parts of load-bearing walls on the 1st overground storey remained. After the fire, the rectory was restored, but in 1865 it burned down again and was newly restored (Figure 2). Up to now, no major construction work has been done in the rectory building. As part of routine maintenance, the roofing and windows were replaced, bathrooms and toilets were gradually installed inside the building, and ceilings were replaced in some rooms. Throughout the whole time of the building's existence, cracks caused by the assumed effects of earthquakes have been repeatedly repaired. The rectory has served its purpose with varying intensity until the present day.

Fig. 2 - Luby Rectory – a) southern elevation, b) eastern elevation

Structure of the rectory building

The rectory building is a two-storey masonry, partially cellared structure with a pitched roof with hips.

The vertical load-bearing structures are founded on strip footings of stone masonry or hand-placed rockfill. According to the information available, the building should be partially based on bedrock and partially on made-up ground and sediments. The underground floor is supported by a stone masonry vaulted structure with a rise of 2.2 m and a span of 4.5 m starting at the floor level. The overground vertical load-bearing structures are made of stone or mixed masonry. The thickness of perimeter walls on the 1st overground storey, including plasters, is ca 1000 mm, on the 2nd overground storey ca 500 mm. The interior load-bearing walls laid out in both directions are 350 and 500 mm thick. The thickness of partitions is 150 and 200 mm. The heads of window sills and door openings in the load-bearing walls are vaulted.

The floor structure over the 1st and 2nd overground storey consists of a wooden beam floor with a soffit (span of ca 3.5 to 5.2 m). The floors in the residential spaces are wooden, the corridors and sanitary facilities are tiled.

The staircase leading from the ground floor to the first floor is a clockwise curvilinear staircase. The stair treads made of stone are mounted in the walls at both ends.

The load-bearing structure of the roof cladding consists of a collar-beam roof supported by purlins. The 160/160 strutting beam is located in each truss. Purlins with dimensions of 140/200 are mounted on 140/200 columns. The columns are anchored by 100/100 braces securing the stiffness of the roof truss in the transverse direction, and 120/120 strips securing the longitudinal stiffness of the roof truss. The 140/200 columns are embedded in tie beams at the floor structure

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level. 120/140 rafters are supported by strutting beams and headers. Headers are used in intermediate rafter trusses. The roof truss load-bearing structure applies tie beams, strips and braces for the horizontal reinforcement of the roof structure. Asbestos slate roof cladding is laid on an overall formwork of planks.

The windows are wooden with insulating double glazing. The entrance and most of the interior doors are massive wooden doors, and new modern smooth doors are fitted in the additionally installed sanitary facilities.

ANALYSIS OF FAILURES IDENTIFIED DURING A PRELIMINARY VISUAL BUILDING TECHNICAL SURVEY

Failures identified during a preliminary visual building technical survey

The preliminary building technical survey included the detection of defects and failures of varying relevance in the building. In accordance with the building and historical development and the effect of degradation processes, traces of increased moisture contents are visible on vertical, mainly perimeter structures, parts of horizontal structures, and the most part of vertical load-bearing and non-load-bearing structures are damaged by cracks. The type of visible traces of increased moisture contents corresponds to the missing or non-functional insulation against rising damp. The results of the survey do not point out any biological degradation and weakening of the wooden members of the roof truss structure and ceilings manifested by increased deformations.

The damage to the rectory building in the form of cracks is evident both on the facade and on the internal load-bearing and dividing structures. There are tensile or, to a lesser extent, shear cracks. Looking at the facade cracks occur most often in the corners of openings. The cracks are found on both overground storeys, they have a slightly inclined shape and interconnect the superimposed window or door openings. A typical crack found in the facade begins on the eaves, continues to the window head on the 2nd overground storey, then passes across the window sill to the window head on the 1st overground storey, and from there it continues via the window sill of the 1st overground storey as far as the masonry footing. Some cracks branch into several other cracks. The crack width varies along the building's height. The narrowest cracks are above the foundations and gradually widen towards the roof. Cracks are found both in surface finishes and the masonry. Cracks occur with varying intensity on all facades, but the building has suffered most damage on the north and east facades (Figure 3).

At the entrance to the building on the 1st overground storey, there are prominent cracks in the right and left corners of the room separating the transverse load-bearing walls from the southern perimeter wall of the building. Cracks of a similar type are found in all connections of internal and perimeter vertical structures on the ground floor. The cracks in the corners of rooms are, in some parts, followed by a number of inclined cracks in the load-bearing walls. There are cracks in the vaulted heads of the openings extending into the ceilings of the floor structure where they often copy the connections of individual structural members of the soffit. The cracks on the 1st overground storey are most often found in the places of connections of structural members – e.g. the connection of the interior and perimeter walls, in the cavettos of the floor structure's connection to the load-bearing walls, mounting of the vaulted strip in the masonry, etc.

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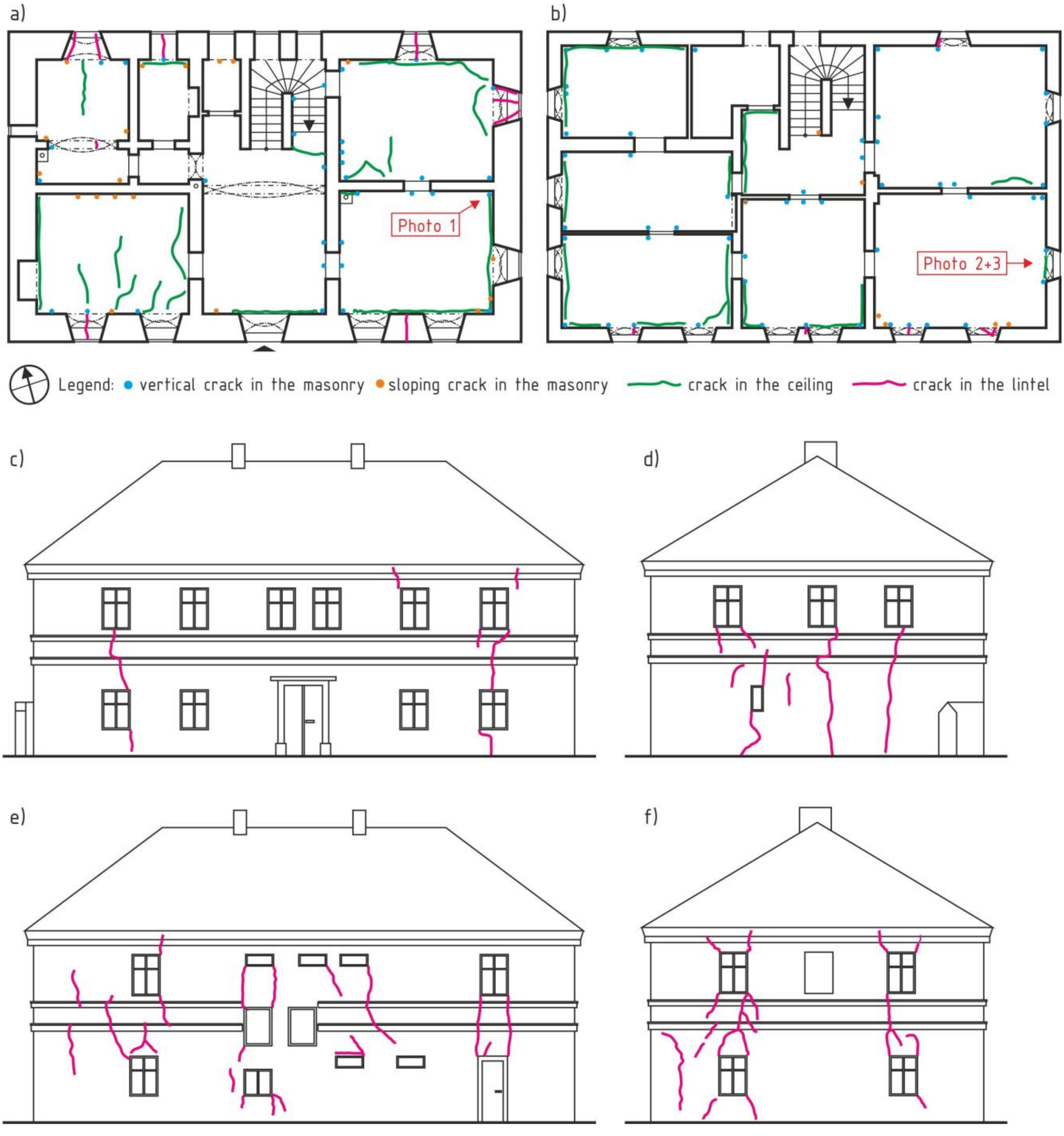

Fig. 3 - Schematic drawings of the rectory with marked failures and points where photographs were taken – a) plan of 1st overground storey, b) plan of 2nd overground storey, c) southern elevation, d) western elevation, e) northern elevation, f) eastern elevation

The type of cracks on the 2nd overground storey is similar to that on the 1st storey. The cracks most commonly occurring there are vertical or slightly inclined cracks in the corners of rooms, cracks in walls and partitions, in the vaulted heads of window and door openings and cracks between individual ceiling elements. On the 2nd overground storey, the rectory building is damaged by more prominent cracks than on the 1st storey. The most prominent - widest cracks are found in the eastern part of the building, where they are 2.5 to 4 mm wide in the perimeter wall. These cracks originate from a horizontal crack separating the floor structure from the perimeter wall, continue all the way to the window recess and then further on across the window sill. Due to the type and location of these prominent cracks, it can be assumed that unless their further development is prevented, they may become the cause of a significant decrease in the structural capacity of the building as a whole in the future (Figure 4).

No reinforcing structures (visible tie cotters of beam or wall ties), or marks of additionally installed stiffening structures were found in the building, neither in the interior, nor in the exterior during the preliminary building technical survey.

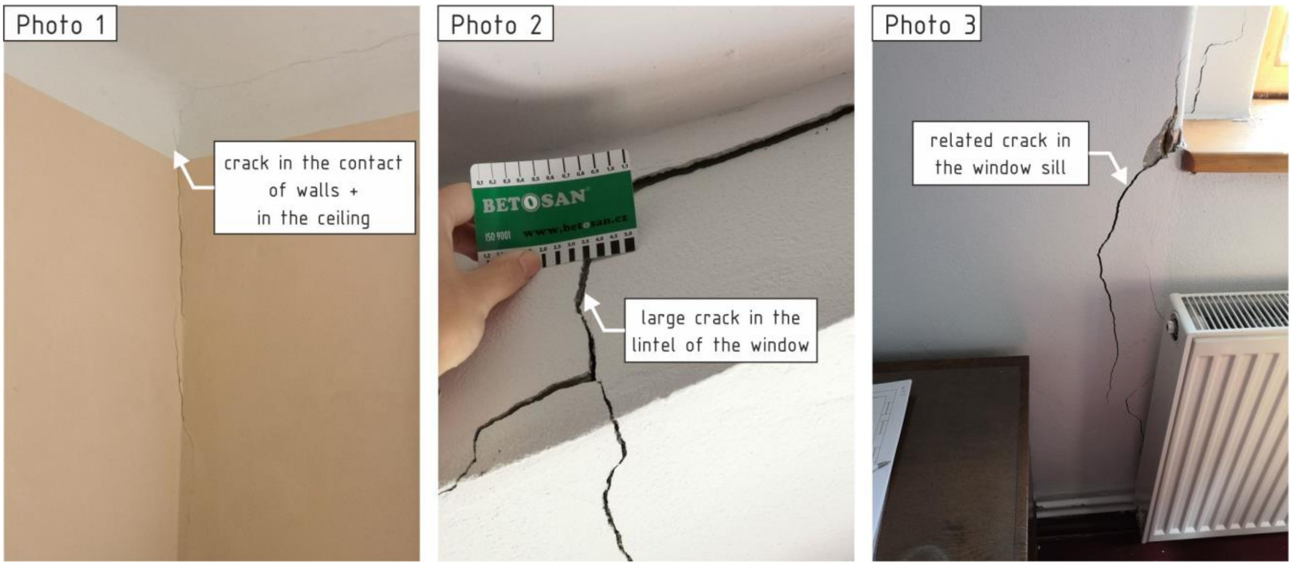

Fig. 4 - Illustration of damage to the building

Numerical model of the structure

To verify the assumption of seismic loading as a possible cause of the detected failures of the masonry structure, a spatial model of the structure was created in the Scia Engineer 19.1 programme on the basis of data obtained during the building technical survey.

The main load-bearing elements of the structure were modelled - load-bearing and reinforcing walls, masonry partitions, floor joists, vaulted arches and truss structures. The walls and partitions were discretized using planar elements with 4 nodes with an edge size of 250 mm. The average number of 1D finite elements on bar members was 10. The total number of elements was 11,077, of which 7,738 were planar elements. Linearly elastic characteristics of the used materials - masonry and wood - were considered. The modulus of elasticity of the masonry considered was 3.1 GPa and Poisson's ratio 0.2. The mechanical characteristics of wood were kept according to the default settings of the computer programme.

Figure 5 displays the computational model and the first mode shape of the structure. The first mode shape corresponds to the theoretically identified first natural frequency of 4.29 Hz. The largest theoretical deformations are located in the eastern gable and correspond to the most extensive damage found on the site. The calculated values of deformations in the places of failures exceed the deformation values for the formation of tensile cracks in the masonry.

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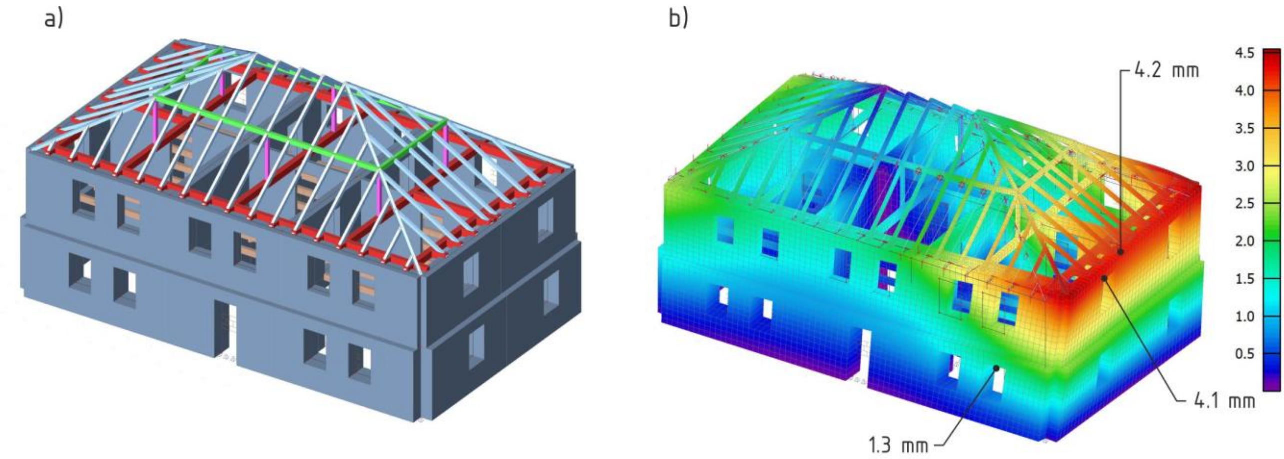

Fig. 5 - a) computational model of the building, b) building's first mode shape

Analysis of the results of a building technical survey and numerical analysis

Due to the type of cracks, we may assume that the change in physical and mechanical properties of the masonry or its components caused by rising damp is not the main cause of cracking. Likewise, the temperature differences in individual structures have a minimal effect on the formation and development of cracks due to their occurrence. The occurrence of cracks in the connections of individual structures indicates their insufficient bonding. The type of structure and its missing adequate reinforcement (at least at the ceiling level) against horizontal loading effects also allow further development and propagation of cracks.

The first mode shape of the structure identified on the basis of numerical analysis and associated deformations confirm the assumption of seismic loading as a possible cause of failure of the masonry structure.

Based on the analysis of the information and findings obtained during the evaluation of the preliminary building technical survey, we may assume that the cause of the occurrence and development of cracks are the effects caused by seismic loads.

DESIGN OF REHABILITATION MEASURES

Due to the probable cause of the occurrence and development of failures in the building and the type of structure (masonry structure of stone and mixed masonry without reinforcing elements or with non-functional reinforcing elements), the most appropriate rehabilitation method is to increase the stiffness of the structure by its additional bracing and prestressing by means of steel ties and prestressing cables, or by carbon fibre-based lamellas. The cracks must be grouted or filled before prestressing the structure. The bracing and prestressing of masonry structures is an effective and reliable protective measure for damaged masonry structures.

A reliable function of beam ties and tie rods, their immediate (not delayed) effectiveness is the most reliable prevention of the appearance or continuing development of tensile and shear cracks in masonry. By using prestressing ropes, cables and lamellas, higher values of masonry prestressing can be achieved compared to steel tie rods. Before mounting tie rods or cables, ropes and lamellas and designing prestressing forces, the stresses in the masonry must be assessed in terms of the prestressing method and the position of e.g. prestressing cables in relation to the prestressed masonry core and the spatial stiffness of the structure (masonry stresses in the direction of masonry bed joints), and it must be ensured that the additional pushing of anchor plates does not cause a significant decrease in the prestressing force and thus a reduction in the masonry prestressing effectiveness. Before grouting the grooves and anchors, it is necessary to check the stability (to avoid a decrease) of the prestressing force in the cables and tie rods.

Additional reinforcement of historic masonry buildings currently mostly applies the bracing of masonry with steel tie rods of circular cross-sections, strip steel, or prestressing patent wires inserted in grooves up to 150 mm deep arranged in both directions of the building (Figure 6). Another solution is an effective and controlled activation of the existing wall or beam ties, tie rods and load distribution anchor plates (e.g. by additional activation of the beam tie attached to the rehabilitated joists).

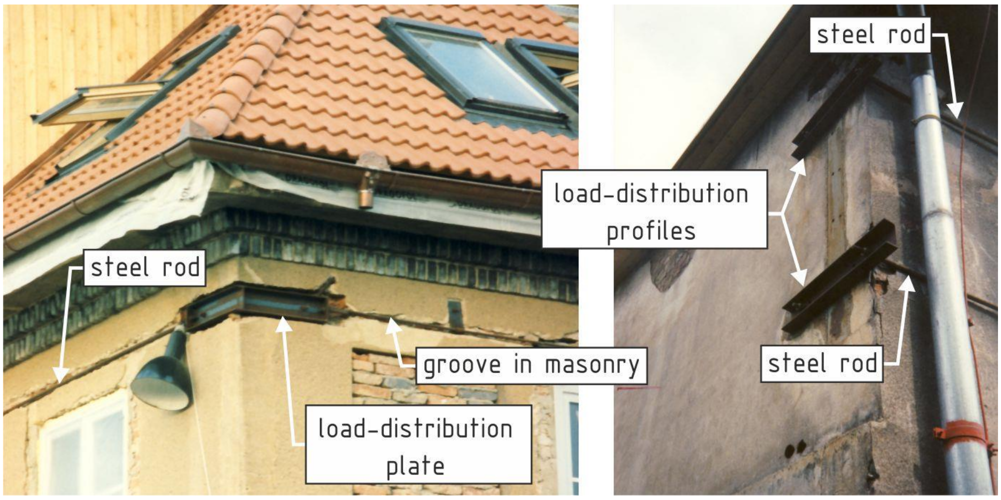

Fig. 6 - a) Bracing walls with circular steel sections, b) temporary bracing of masonry [10]

In some cases, a newly executed bond beam of reinforced concrete or prestressed concrete may serve as “passive” static protection (stiffening) (e.g. during a complex roof truss replacement, extension of a building, etc.).

Rehabilitation with CFRP lamellas

Damaged masonry buildings with insufficiently functioning tie rods and beam ties, or with the absence of wall and beam ties in some historic buildings can be reinforced with lamellas based on carbon or aramid fibres fitted in their end parts with special metal anchoring elements enabling the activation or pre-tensioning of the lamellas to the required force. Steel load distribution plates or special prestressing devices must be mounted in the anchoring parts of the lamellas and the masonry must be reinforced (Figure 7) [11, 12].

A lamella based on carbon or aramid fibres can be attached in a special metal anchor sleeve of enclosed shape with a wedge-shaped hole for passing the lamella through it, which allows its anchoring by metal wedge-shaped plates with a special surface treatment on both sides – a roughened surface at the contact with the lamella and grooves at the contact with the enclosed anchor sleeve similarly provided with grooves enabling reliable clamping of the lamella and preventing its slipping (slacking). The lamella-metal anchor plates contact surfaces should be treated with epoxy resin before adjustment. The lamellas can be prestressed by means of prestressing (spacer) screws fastened to the sleeves using so-called “torque” wrenches.

Lamellas (beam ties) can pass along wooden floor beams or steel floor joists, or on the upper surface of subfloor planks, and must be secured in position by clips at a 1.5 - 2 m spacing. Depending on specific conditions and in accordance with the requirements for fire resistance or protection against mechanical failure, the lamellas based on carbon or aramid fibres can be protected by special fire-resistant cover strips, fire-retardant coatings, etc. In this modification, the prestressing by a lamella is imposed into the structure by a “solitary” force acting on both ends of the lamella in the place of the anchor sleeve and the load distribution plate. Securing the required effectiveness of prestressing cables, prestressing steel tie rods and carbon lamellas requires that prestressing should be applied to the existing masonry structure along the entire length of the prestressing tie rod, lamella, etc. (Figure 8). In cases where a prestressed lamella, fixed by the prestressing device and laid in a shallow groove is coated with structurally effective plaster, adhesive sealant, an adhesive layer, etc. and the prestressing device is only released after the required strength of this "covering and fixing" layer has been reached, the prestressing force can be applied along the entire length of the lamella (Figure 8). This solution usually removes extreme stresses in the place of prestressing devices and anchor plates [10].

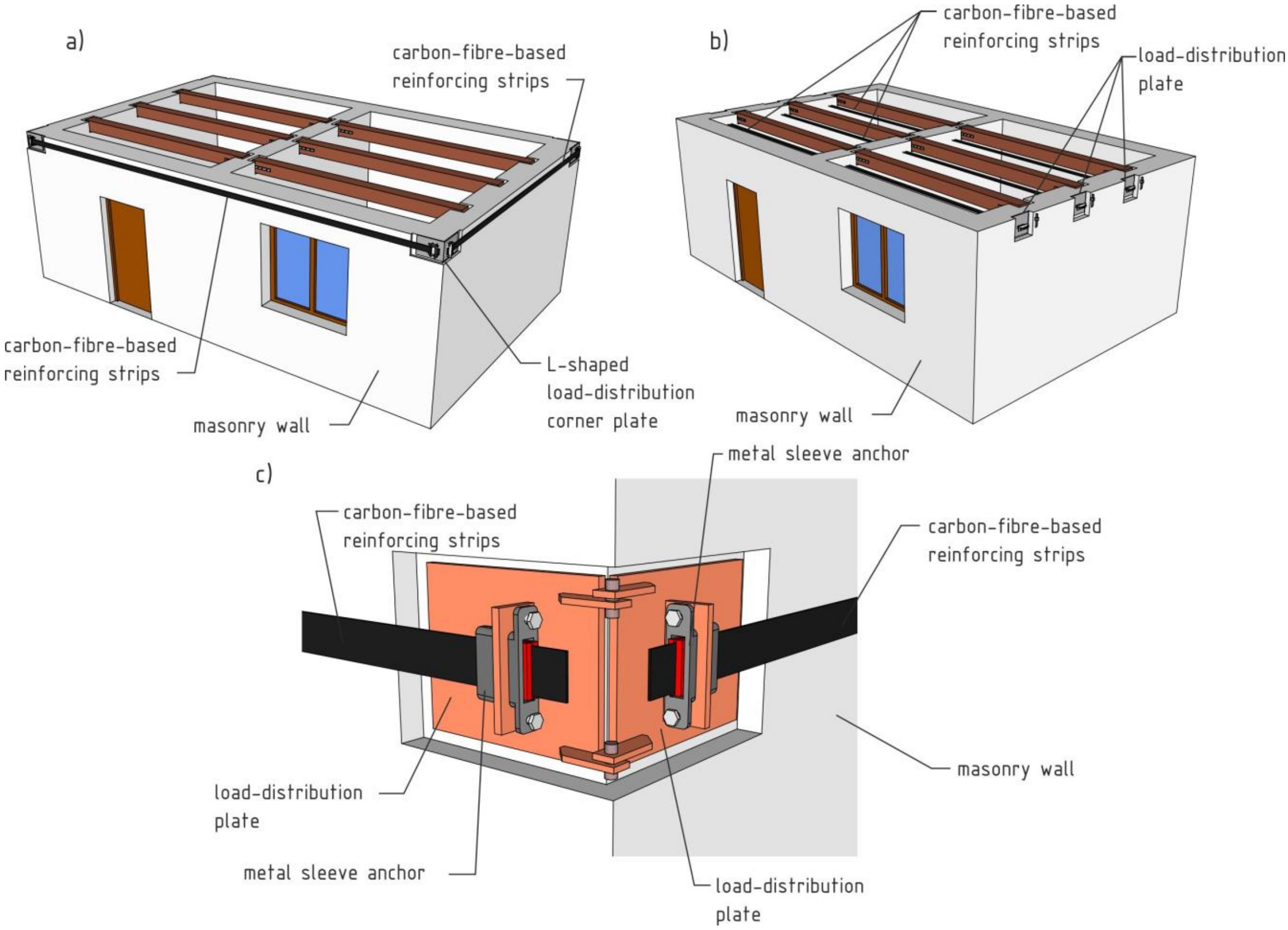

Fig. 7 - Reinforcement of damaged masonry buildings with insufficiently functioning tie rods and beam ties or with their absence by means of lamellas based on carbon or aramid fibres

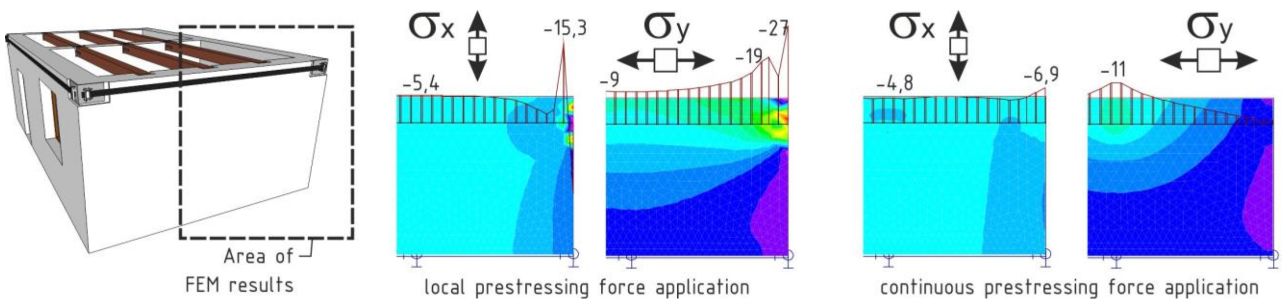

Fig. 8 - Stresses in masonry in the end area of prestressing members

The magnitude of prestressing forces must be identified by a structural calculation in accordance with the time pattern and magnitude of horizontal normal stresses and shear stresses and the condition of masonry.

Special attention must be paid to the treatment of masonry and surfaces in the places of anchor plates (corrosion protection, damaged integrity of surface treatment, effects of moisture and temperature near the anchors, etc.).

Lamellas 60 - 100 mm wide and 2 to 4 mm thick fitted with special anchoring and prestressing elements can be applied directly onto undamaged, treated and ground masonry to create a smooth surface without protrusions. In the case of masonry with damaged surface layers, after removing the damaged and incohesive parts of the masonry, the lamellas are applied onto the masonry surface levelled with a thin layer of polymer modified cement or lime-cement mortar, or onto the treated surface of damaged masonry after its previous grouting and the repair (sealing) of cracks using a similar method as in the case of steel cables and tie rods. Depending on the depth of the lamella sinking in the masonry, the stresses and stability of the wall must be assessed taking into account the masonry thickness, prestressed lamella position, distance of the walls laid out perpendicular to the prestressed wall, the prestressing force magnitude and, as a rule, the large eccentricity of prestressing forces. Also, the perimeter wall must be stabilized against buckling by transverse walls or struts (e.g. pipes into which transverse expansion lamellas – collets are inserted). Proper remediation requires the wall to be braced so that the resultant bracing force passes through the core of the cross-section, i.e. the tie must be placed at the outer and inner face of the wall. For small prestressing forces (small forces caused by the effects of seismicity), it is possible to place prestressing elements only at one wall surface.

The anchoring areas of prestressed reinforcing lamellas must be fitted with metal load distribution plates, mounted on the reinforced masonry, modified for mounting the anchoring metal elements at the ends of reinforcing lamellas.

In cases of active prestressing of lamellas to higher tensile force values, the building must be prestressed around the perimeter by means of lamellas anchored in “L”-shaped corner metal load distribution plates located in the corners of the building at one height level. The prestressing must be performed gradually by alternately applying the prestressing force in both directions (see Figure 7).

Depending on specific conditions and in accordance with the requirements of fire resistance or protection against mechanical failure, the lamellas based on carbon or aramid fibres can be protected by special cover strips made for this purpose, fire protection coatings, plaster or they can be laid in a groove filled with a polymer modified cement screed applied after the mounting of lamellas.

Another possibility of horizontal reinforcement of e.g. perimeter masonry is the stiffening (strengthening) of masonry by means of lamellas based on carbon fibres 40 - 100 mm wide, inserted in a widened groove in the bed joint, or in a groove cut outside the bed joint (in irregular masonry without a continuous bed joint) deeper by min. 20 to 40 mm than the width of the reinforcing lamella. After cutting and cleaning, the horizontal groove is filled with a special mixture based on polymer modified cement with an adequate consistency to enable pushing the lamella into the joint. In another case, the lamella can be inserted into an open joint, and subsequently, after the joint is sealed and the lamella prestressed, the joint is injected (sealed) with a polymer modified cement mixture.

Reinforcing lamellas can be installed in any place depending on the structural requirements, the occurrence of cracks and the extent of damage (e.g. in every 3rd bed joint, in thirds of the masonry height, etc.).

The advantage of the proposed masonry reinforcement, in addition to the variability of the distribution of lamellas along the height of the masonry (building), is mainly limited disturbance of the historic masonry, preservation of the original appearance and dimensions of the masonry structure – after the installation of reinforcing carbon lamellas into the bed joints or horizontal grooves and the mounting of metal anchoring elements, the masonry can be coated with plaster in the original thickness, quality and composition (does not require the execution of demanding covering layers).

Rehabilitation of the rectory building with lamellas based on carbon or aramid fibres

A numerical analysis of reinforcing (prestressing) of the building using lamellas based on carbon or aramid fibres is performed as a part of building's rehabilitation design. The reinforcing was introduced into the computational model of the building in two steps. In the first step, the reinforcing of perimeter walls at the level of the ceiling of the1st and 2nd floors and reinforcing of the building's corners were modelled. Figure 9a shows the first mode shape of the building with reinforcement. The maximal values of deformation decreased to 70 % of the values of deformation of the building without prestressing (see Figure 5). The calculated first natural frequency increased to 5.12 Hz. In the second step, in addition to the reinforcing of perimeter walls, the reinforcing of inner walls was introduced. The character of the first mode shape has slightly changed and the largest values of calculated deformations also occur on the opposite side of the building (Figure 9b). The maximal value of deformation reaches approximately 60 % of the deformation of the building without prestressing and it decreased by 15 % in comparison with the value of deformation for reinforcing only of the perimeter walls. The theoretically determined first natural frequency of the building with reinforcing of perimeter and inner walls was 6.22 Hz. The numerical analysis confirmed the positive effect of prestressing on increasing the rigidity of the building.

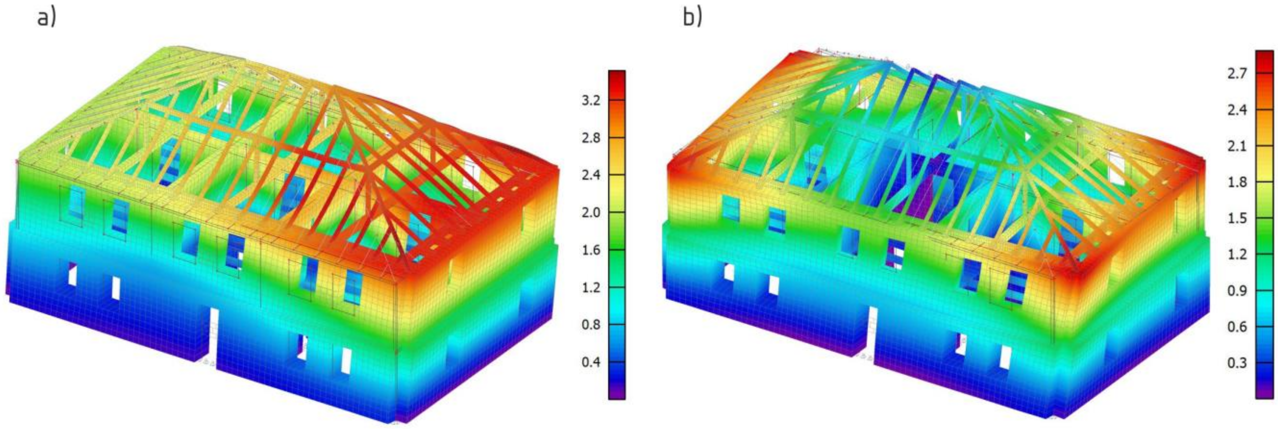

Fig. 9 - Building's first mode shapes: reinforcing of perimeter walls (a), reinforcing of perimeter and inner walls (b)

The proposed rehabilitation method - increasing the stiffness of a two-storey historic rectory building in Luby damaged by the effects of natural seismicity - with the use of lamellas based on carbon, or aramid fibres is displayed in Figure 10.

The rehabilitation method respects the results of executed numerical analysis. Additional reinforcing prestressing elements are designed at the level of the ceilings of 1st and 2nd floor in the perimeter walls and inner load-bearing walls. Due to the absence of beam anchors, the reinforcing system is extended by the reinforcement of wooden beam ceilings in places of missing or non-functional anchors using lamellas based on carbon or aramid fibres and resins. The use of prestressing elements was verified in a performed numerical analysis.

The diagram shows the position of prestressing lamellas, the location of anchoring and load distribution elements. The position of individual elements of the additionally executed reinforcing system of the masonry structure is chosen so as to minimize the interference with the historic structure, but, at the same time, maximize its stiffness gain.

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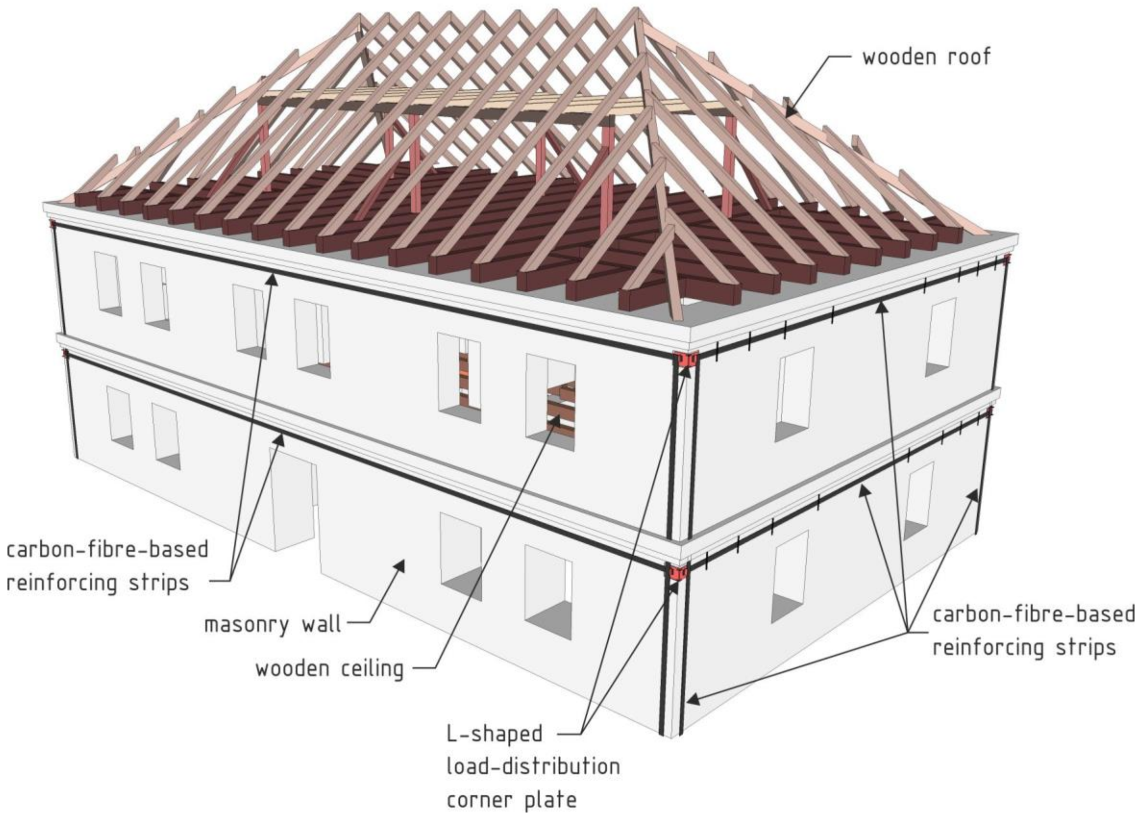

Fig. 10 – Scheme of the rectory building with a marked layout of FRP lamellas and steel anchoring and load distribution sections

SUMMARY

The preliminary building technical survey, the analysis of its results and the numerical analysis allowed identifying the most likely cause of damage to the historic rectory building in Luby near Cheb. The results obtained during the previous research within the NAKI research project were used in the conceptual design of the rehabilitation of a historic building damaged by the effects of natural seismicity consisting in increasing the building's stiffness. Compared to the reinforcement of the structure with steel prestressing elements, the properties of the used prestressing elements enable reducing the negative impacts of the designed rehabilitation method on the historic structure.

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REFERENCES

[1] Schenk, V. and Schenková, Z.: Maps of Seismic Zones in Recent Czech National Codes. Exploration Geophysics, Remote Sensing and Environment, IV.2/97, ČAAG-Praha, 9-11, 1997

[2] Monument – databáze památek NPU, http://monumnet.npu.cz

[3] Geofyzikální ústav Akademie věd České republiky, Seismické oddělení

[4] Celep, Z., Icncecek, M., Pakdamar, F.: Structural and earthquake response analysis of the Muradiye mosque, 14 WCEE, Beijing, China Conference: Proceedings of the 14th World Conference on Earthquake Engineering, 12.-.17.10.2008,
[5] Borri, A., Corradi, M., Vignoli, A.: New materials for strengthening and seismic upgrading interventions, International Workshop Ariadne 10, Arcchip, Prague, Czech Republic; 04/2002
[6] Ravikumar, C.S., Thandavamoorthy, T.S.: Application of FRP for strenghtening and Retrofitting of civil engineering structures, International Journal of Civil Structural, Environmental and infrastructure Engineering, Research and Development (IJCSEIERD), ISSN (P) 2249-6866, ISSN (E) 2249-7978, vol. 4, Issue 1, feb. 2014, p. 49 – 60
[7] Makovička, D., Makovička, D. jr.: Response Analysis and Vibroinsulation of Buildings Subject to Technical Seismicity, In: Earthquake Resistant Engineering Structures VII. Southampton: WIT Press, 2009, pp. 197-205. ISSN 1746-4498. ISBN 978-1-84564-180-1
[8] Elghazouli, A. (Ed.). Seismic Design of Buildings to Eurocode 8. 2009. London: CRC Press
[9] Lorant, G., Seismic Design Principles, dostupné z: https://www.wbdg.org/resources/seismic-design-principles [citováno 2018-03-21].
[10] Witzany, J. a kol., Obnova rekonstrukce staveb. Poruchy, degradace, sanace, Praha: CTU. Czech Technical University Publishing House, 2018. ISBN 978-80-01-06360-6.
[11] Witzany, J. Zigler, R., Předpínací kotva pro zpevnění nosného zdiva, Czech Republic. Utility Model CZ 31090. 2017 (in Czech)
[12] Witzany J., Čejka T and Zigler R.. Stiffener for bracing of masonry buildings particularly historical. Utility design, Úřad průmyslového vlastnictví, 23023. 2011 (in Czech)