Strengthening of Masonry Structures on the Undermined Area by Prestressing

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The article deals with determining of causes of cracks of walls and vaults in existing church in Staříč in the Frýdek – Místek (Czech Republic) and determination of its remediation of measures. On the basis of this case, the laboratory equipment was designed and built at Faculty of Civil Engineering VSB – TU Ostrava for measurement of tri-axial stress-strain conditions in historical and existing masonry. The laboratory equipment should permit the measurement of deformations and stress state in the masonry in the ratio to reality 1:1. Mathematical modelling of brick corner is based on finite element method using software ANSYS and then the results are compared with results of laboratory tests. On the basis of these results it should be possible to improve the models, approach closer to the accurate models and at the same time use simple procedure for design of pre-stressed masonry.

Key words: *Historic Structures, Undermined Territory, Masonry, Pre-stressing, FEM, Numerical Simulation*

Introduction

Although coal mining is currently in the downturn in Ostrava region, mining is still ongoing in some coal seams and in the future their mined-out is planned in technical and economical acceptable way. Besides normal civil and industrial buildings that are usually secured against mining effects, some historically valuable buildings are affected as well [1], [2]. The article gives an approach to monitoring of effects of past coal mining and also future mining activities regarding the building of the St. Cross church in Staříč in Frýdek-Místek region.

Fig. 1. Spatial model of the church.

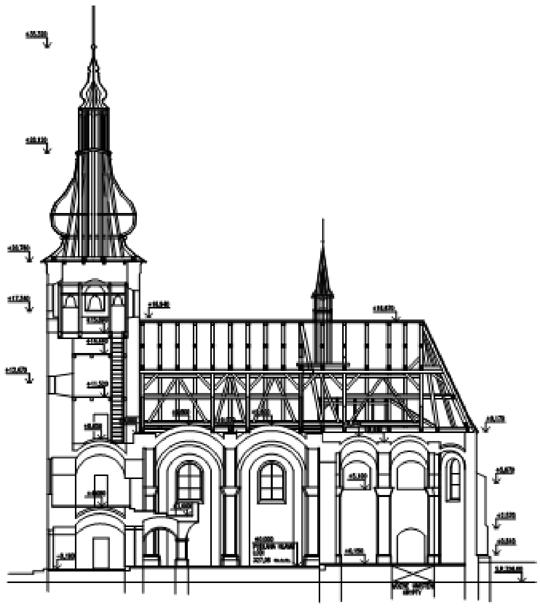

Fig. 2. Longitudinal section – Church protection.

The building of the Roman Catholic parish church in Staříč is the Baroque building with rococo equipment. The church is built on the historic core. The first written record of the parish in Staříč is dated to 1290. The original church was made of wood; the present building was constructed of stones and bricks in the beginning of the 16th century [3].

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The ground plan dimensions are about 30 m in the longitudinal direction (in the church axis); the width of the object varies from 16.5 m in the entrance to 19 m in the rear. The total height is 38.3 m.

According to building survey [2], the walls of the main aisle, the main chapel and the tower are built from quarry stones. The stones are not hewn and they are laid to lime mortar. The average thickness of the main enclosure walls is 1.0 m. The walls of additional parts (sacristy, side chapels and oratories) are built from mixed stone and brick masonry.

The floors are made as vaults – pendentive vaults. The vaults are supported by peripheral load-bearing walls and vault arches. The joist floors above the oratories at the sides of the chapel are made with flat plaster ceiling. The building is stiffened with steel reinforcing collets placed in the supporting walls at the floor level.

Detected faults and their causes

Within the structural and technical survey, the building was measured in detail and inspected. Detected faults and cracks were documented in detail, description of the failures and photographic documentation are a part of the opinion.

On the outer peripheral wall, there are vertical cracks located which extend through the entire thickness of the wall. Cracks are evident even in the most sensitive elements of the object (in the vaults inside the church). Progress of cracks on the inner side of the walls in essence corresponds to the course on the outside. In the same way, belts of vaults above windows are also damaged in the middle of the rise. The walls between the nave and side extensions show similar way of damage - shear cracks are ongoing in the vertical and oblique direction.

Monitoring of cracks is already in progress using gypsum targets which are fitted on the plaster. According to the target from November 1998, originated cracks showed that the movement continues in cracks and that they are still active. Minor cracks also occur in other parts of the building. The most of minor cracks is found in the belts of vaults.

Horizontal isolation between the foundations and masonry enclosure wall has not been found. Due to the age of the object, isolation is not expected. On the level of ground floor, there are traces of wetting and capillary action of water. Although it is not a typical mining damage, the influence of undermining could make the situation worse because masonry is cracking and pores are growing.

The upper surface of the vaults above the main aisle did not show visible cracks. In many places the top surface is very uneven and side walls of the parapet walls are without plaster.

Effects of mining activity on the surface of the area of interest are a part of mining expert opinion from October 2001. Mining opinion deals with influence of mining according to recent subsidence measurements carried out at the end of May 2001 and with the views of planned mining until 2020.

Measurement of subsidence was carried out on stabilized points along the perimeter of the church. On the basis of measurements, numerical analysis based on the theory of prof. Knothe was performed. The results were compared with measured values. According to modified input parameters, forecast (until year 2020) of subsidence under the church was performed. The results of the analysis show that the projected overall declines of the church reach a value of about 1.2 m.

Deformation Load

Deformation load generally causes major internal forces within the building structures. Below are most frequent deformation impacts: temperature, creeping and shrinking, undermining, flooding, pre-stress.

Fluctuation of daily/yearly temperatures influences all building structures. Under normal conditions, if limit dimension of expansion units are followed, such fluctuation does not cause cracks to arise. Major impacts result from undermining and uneven subsidence caused by water-flooded underlying rock [4], [5], [6].

When extracting a seam by means of a reasonably wide working (working face), changes also occur in geostatic/tectonic state of stress within neighbouring rock massif. The changes are accompanied with deformation and shift of rock from the overlying rock into the mined-out area [7], [8]. In case of long mining workings such as adits or roads, the impacts are not in fact evident beginning from the low depth - thanks to arch action of the rock. In case of space mining workings, a subsidence through will appear after a certain time, depending on the excavation depth, geological structure of the overlying rock, seam thickness, and excavation method [9], [10]. The depth and layout of the subsidence trough depend mostly on the depth (h) and thickness (m) of the seam to be extracted, and limit angle (μ) for surface extraction. In the Ostrava - Karviná coal basin, the limit angle is about 65 degrees [11]. The volume of the subsidence trough depends also on the extraction method which is expressed by the extraction factor (a), being 0.8 to 0.9 if collapse extraction is used. The subsidence through depth (s) is greater with greater seam thickness and lower seam depth under the surface.

The subsidence trough consists of an internal quiet part, and boundary parts. The depth of the internal quiet part is almost identical as the subsidence depth (s), while the boundary parts are of vital importance for the proposed protective measures taken for the ground building.

In order to describe the landscape deformation intensity in the subsidence trough boundary parts, the mining industry uses following geometrical quantities: s - subsidence [mm], v - horizontal shift [mm], i - inclination [rad], R - radius of bending [km], ε - horizontal relative deformation [-].

The inclination (i) and horizontal shift (v) have peaks in the subsidence line inflection point (s) above the working face edge. The horizontal relative deformation (ε) and the landscape bending (ρ = 1/R) reach the maximum at about + 0,4 r from the working face edge.

Most dangerous for the ground buildings are the horizontal relative landscape deformations (ε). They are positive, if above the subsidence line inflection point (landscape elongation), or negative, if under the subsidence line inflection point (landscape compression).

The calculation of the horizontal deformation of terrain was performed in two stages with different coefficients of mining. As it results from the measured values, the church was compressed in both directions previously. As a result of further mining, on the west side tensions will occur in the longitudinal axis while the compression will decrease from present values of compressive strain:

$$\varepsilon_{\max} = -1.0 \cdot 10^{-3} \; to \; 0.8 \cdot 10^{-3} \tag{1}$$

on the values of strain on the east side:

$$\varepsilon_{\max} = -0.6 \cdot 10^{-3} \; to \; 0.2 \cdot 10^{-3}. \tag{2}$$

In the transversal direction the compressive strain in the range after a slight decrease returns to the previous value.

$$\varepsilon_{\max} = -0.6 \cdot 10^{-3} \; to \; 0.3 \cdot 10^{-3} \tag{3}$$

On the basis of mining report, measurements results of the current state, engineering-geological survey and structural-technical survey, it is possible to determine the cause of cracking and to design a concept of static security for future exploitation.

From the character of vertical cracks that extend over the entire height of the perimeter walls in the longitudinal direction, it can be deduced that damage of masonry occurred due to tensile stress. It was caused by compressive strain of terrain due to undermining effects. According to mining report, the object is located in the area of negative strain of terrain when compressive stresses are formed.

The evaluated measurement shows that the object is in torsion along the longitudinal axis of the church. This brings similar character of cracking of the walls but these cracks are caused by shearing forces.

Rehabilitation of Cracks

The masonry with cracks can be rehabilitated by injecting the polyurethane resin mass. The tensile strength of this mass exceeds considerably that of the masonry as well as that of the concrete. Consequently, other distortion, if any, arises out of the injected crack.

Any proposal for injection must be however supported by a detailed analysis of reasons and correction actions. Otherwise, the cracks will continue appearing in other cross-sections.

A number of different polyurethane injection systems can be used. A particular resin and resin type must be proposed considering the injection area, viscosity, necessary strength, ageing resistance, setting time, environment temperature, oil resistance, and chemical resistance. The resulting mechanical and physical properties of the injection mass or injection environment often depend on the foam level as achieved [12], [13].

If the load-carrying capacity of the underlying rock is not sufficient, the deformation properties of the rock can be increased by the injection. It is however essential to keep in mind that the increase in the underlying rock stiffness results also in re-distribution of the internal forces within the foundation - even with the same load.

The underlying rock injection can not only increase the foundation soil deformation module, but also fill in the cavities between the foundation and the underlying rock. It is how-ever necessary to monitor the injection pressure in order to avoid undesirable deformation of the structure under rehabilitation. Otherwise, it is possible that the components of the impaired structures with poor load-carrying capacity, such as floors or week masonry, will be destroyed.

The injection mass must be selected, considering its suitability in terms of environment resistance and environment durability. The quality of the mechanical and physical proper-ties of the injection mass should be better than those of the structure under rehabilitation [14], [15].

Increase in Structure Stiffness

When rehabilitating existing cracks or eliminating possible impacts of mining activities or floods, it is essential to increase the space stiffness of the building. Pursuant to many recommendations and comments to ČSN 73 0039 [16], it is desirable to employ the stiff structure systems for following buildings, especially:

- the buildings which are sensitive to changes in the structure's geometrical shape caused by bending impacts and horizontal landscape deformation - if additional correction or step-by-step rectification is not possible.
- the buildings the layout of which enable the undermining force impacts rather than the deformation impacts to be transferred more easily - it would be more difficult to size these buildings in order to "release" such impacts. Typical examples include massive cast-in-place foundation structures, slab blocks, masonry, other tower-like building, cast-in-place or fabricated carrying structures for major effective load, or closed underground structures for sumps, channels, or collectors.

Horizontal Reinforcement of Buildings

The additional horizontal reinforcement of walls in a masonry building ranks among most typical protections against horizontal landscape deformation or bending impacts. For that purpose, following solutions are available: reinforced-concrete rings with the closed layout, steel tie members, or pre-stress ropes or cables.

The reinforced concrete rings are usually used in ceiling structures. First, a groove is cut out into the masonry and stud connectors are installed. The longitudinal and transversal tensile forces are then transferred through the reinforcement of this ring. A disadvantage consists in partial weakening of the wall, this being the case of subtle masonry components mostly.

The steel tie members are often installed on-to the outside/inside face of the enclosing walls. Advantages include easy erection, while disadvantages consist in poor efficiency (in case of decaying impacts - too extensive prolongation of the tie members), necessity to provide corrosion protection, and the appearance.

The pre-stress which arises in the building under rehabilitation using the pre-stress reinforcement is highly effective - from the point of view of the influence, and from the method itself. The pre-stress cables are generally used within the building at the level of the ceiling structure. At the level of the foundations, the cables are laid in an additionally extended continuous footing which is combined with the original one [17], [18], [19].

Reinforcement and Pre-stressing of Foundation Structure

Reinforcement of the foundation structures and additional pre-stressing of the bearing walls in their foundations is of key importance for elimination of tensile stress in the carrying system and creation of other cracks, if any. Before pre-stressing, it is however essential to reinforce the existing foundation made from the rubble masonry. For that purpose, the injection is placed into the joint between the rubble masonry, and the new concrete foundation ring is combined, using the stud connectors [20], [21].

The purpose of the backfilling of the side surfaces of the foundations with materials with lower strength/deformation parameters is to restrict impacts on foundation side walls.

If no detailed data about the strength/deformation parameters are available for the backfilling materials, values listed in ČSN 73 0039 [16] should be used.

Use of piles with the aim to increase the load-carrying capacity of the foundation on the undermined area entail other little-investigated-into risks:

- positive horizontal deformation of the landscape (elongation) results in partial loss of the skin friction
- vertical relative deformation in both directions have been measured in past years in the underlying rock of the buildings. The deformation of this kind is more pronounced that the horizontal relative deformation. The vertical deformation creates negative skin friction, damaging possibly the designed function of such pile.

If the combined pre-stress continuous footing is used, it is not necessary to provide pile foundations generally, or to increase the load-carrying capacity of the underlying rock by injection. The extended continuous footing decreases considerably the contact stress in the footing bottom and, in turn, the friction be-tween the foundation and underlying rock [22], [23].

If the flood-affected buildings are to be rein-forced, the piles represent most suitable and effective tool, increasing the resistance of the foundation structure against erosion of fine-grain articles [24], [25].

Space Stiffness

As the buildings are frequently subject to unfavourable torsion loads caused by the uneven subsidence, it is impossible to reach the required stiffness by typical rehabilitation, this means by mere reinforcement of the foundations. During the torsion, the walls are shear loaded. This results in typical vertical cracks along the wall thickness in regular distances. The resistance of the existing enclosing walls against the shear forces can be increased by pre-stressing the walls and foundations and by injecting the damaged cracks.

Regulations, standards and research of pre-stressed masonry buildings

The concept of pre-stressing masonry dates back to 1825. There was the method of additional pre-stressing used for tunnel on the river Thames in England but in the 20th century were applied procedures based on static analysis of masonry structures. According to this analysis was presented (in the 1943) the first standard dealing with pre-stressed structures in Switzerland, in the USA in 1966 and in the Great Britain in 1985. The British Standard dealt with mostly general principles of pre-stressed concrete and was accompanied by recommendations for use in masonry.

According to Czech and German research [26] it is possible to choose the value of the pre-stressing force ranging from 0.10 to 0.15 of the strength of masonry perpendicular to bed joint. The strength of masonry depends on the percentage of filling of vertical joints with mortar. It is a similar according to [27]. There is selected pre-stressing strength as 1/10 of the masonry compression strength perpendicular to bed joints. In the work [28] is the compressive strength of masonry in parallel to the bed joints can acquire values from 0.1 to 0.85 times the strength perpendicular to the bed joints. According to the results of the work are the values range from 0.1 to 0.25 of the strength of masonry perpendicular to bed joints. This range is dependent on sufficient information about masonry and especially its quality filling of vertical joints with mortar. The value of masonry with a well-filled joints, which should not be exceeded, is of 0.7 to 0.8.

In the existing technical standards (Czech Republic) and literature completely is missing information used to calculate the tensile strength of masonry or structural analyzes of anchor plates. On the base this case was designed and built the laboratory equipment at Faculty of Civil Engineering VSB – TU Ostrava for measurement tri-axial stress-strain of masonry [29], [30]. Experiments should intended to serve for determine appropriate pre-stressing forces so as not to exceed the strength of masonry in place concentric load from pre-stressing forces [31], [32]. Pre-stressing forces in the experimental measurement of deformations described in this article are chosen safely with regard to the quality of filling of joints with mortar from 10 to 80% of the compressive strength of masonry perpendicular to bed joints. The tests simulate the behavior of masonry reinforced by pre-stressing wire ropes at the moment of introduction of pre-stress. These tests are therefore short-term [33], [34].

Laboratory equipment for testing of tri-axial stress

The dimensions of steel structure of laboratory equipment are 900 x 900 x 1550. In the structure is the brick corner about height of 870 mm. The thickness of brick wall is 440 mm. The masonry was built from CP 290x140x65, P15 bricks and from lime mortar, mixed with sand in the ratio of 1:4. The value of average compressive strength of bricks is 12.87 MPa. The value of mean compressive strength of masonry element is 9.9 MPa. Bricks density is 1535 kg/m^3, modulus of elasticity is 4.2 GPa and Poisson's ratio is 0.15. The average compressive strength of the mortar is 0.429 MPa. Mortar density is 1740 kg/m^3, modulus of elasticity is 0.03 GPa and Poisson's ratio is 0.2.

The tested masonry is considered a part of the existing structure and therefore the procedure in calculation of the characteristic strength of masonry follows the standard [35] - Assessment of existing structures, which refers, in determining the strength characteristics, to the previously applicable standards, for the masonry for example to the already invalid pre-standard. The resulting characteristic compressive strength of masonry perpendicular to the bed joints is 1.437 MPa [36].

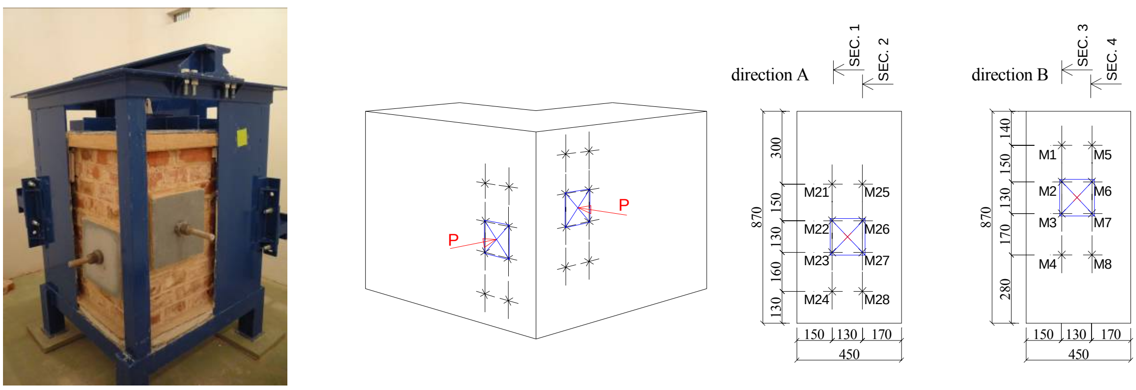

Fig. 3. Schematic layout of measuring sensors in the direction A and B.

During the brickwork two pre-stressing bars were inserted in the masonry at different heights see Fig. 3. Each pre-stressing bar was marked according to the direction, see Fig. 3. In the A direction, it was place at the height of 355 mm, in the B direction, it was placed at the height of 515 mm. The upper part of the structure was aligned by a layer of mortar and steel plate with a thickness of 12 mm with welded steel reinforcements. On the pre-tensioning bars were fitted with steel anchor plates on a layer of mortar.

Vertical load was introduced by a hydraulic cylinder, which was placed between the distribution plate and the I-profile bolted to the laboratory equipment. The experiment was loaded with vertical load of 0.1 MPa.

Pre-stressing force (F), see in the Table 1, was introduced in the pre-stressing bars also by hydraulic cylinders through the anchor plates with dimensions of 150 x 150 mm and a thickness of 10 mm. Measured deformations were recorded using potentiometric sensors attached to laboratory equipment. A total of eight sensors were attached in each direction, sensors M21 to M28 in A direction and sensors M1 to M8 in B direction, see Fig. 3. The experiment was loaded gradually with a pre-stressing force from 10% to 80% of the masonry compression strength perpendicular to the bed joints. At first in the direction B and then in the direction A. Table 1 shows the input load values of the masonry. The first column shows the loading process, the second shows stress values (σ_K) in the anchorage area, the third column shows the pre-stressing force (F) and last column shows the value of modulus of elasticity (E_M).

Tab. 1. Input values for measuring and numerical modeling.

150 x 150 x 10 mm; f_k=1,437 MPa							
Loading process	**σ_K** [MPa]	**F** [kN]	**E_M** [GPa]	**Loading process**	**σ_K** [MPa]	**F** [kN]	**E_M** [GPa]
10%	0.144	3.23	3.00E-02	**50%**	0.719	16.17	1.28E-02
20%	0.287	6.47	1.50E-02	**60%**	0.862	19.40	1.28E-02
30%	0.431	9.70	1.28E-02	**70%**	1.006	22.63	1.28E-02
40%	0.575	12.93	1.28E-02	**80%**	1.150	25.87	1.28E-02

The resulting deformations are in A direction on the Fig. 4 (as the diameter of Sec. 1 and Sec. 2) and in B direction on the Fig. 5 (as the diameter of Sec. 3 and Sec. 4). The x-coordinate contains values of deformations with a negative sign induced by the pressure of anchoring plate on masonry. On the vertical axis there are elevation coordinates of the location of individual sensors according to Fig. 3.

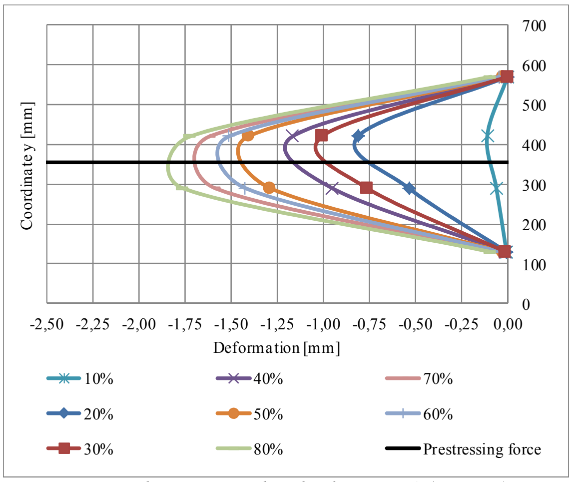

Fig. 4. Values measured in the direction A (average).

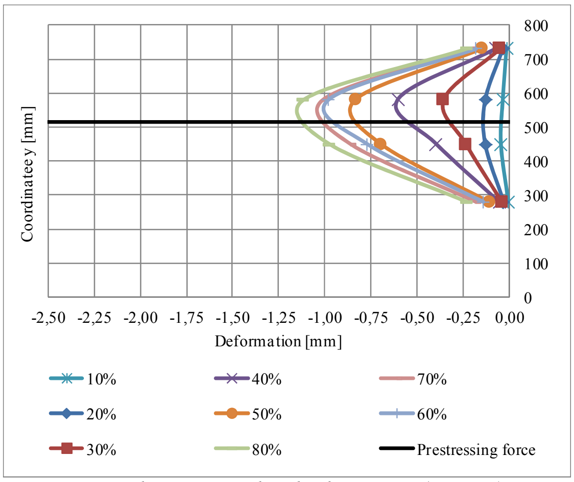

Fig. 5. Values measured in the direction B (average).

As is evident from Fig. 4 and Fig. 5, the shape of deformation of masonry in the A direction at the place of pre-stressing bar corresponds to the stress concentration just below the anchoring plate, while above and below the anchoring plate the deformations are smaller. The results were compared with measurements of the same sample with the opposite order of pre-stressing, i.e. first in the A direction, then in the B direction. The results were similar for both directions, and therefore this comparison implied that the sequence of introduction of pre-stressing forces is not as important as the elevation of anchoring plates.

Numerical simulation of tri-axial stress

Modelling of masonry structure is performed in the ANSYS application, based on MKP. Numerical model is created by means of a so-called micro-model. Micro-model means rendering of actual arrangement of masonry elements [37]. These elements correspond to the bedding of bricks during bricklaying of the structure, including contact and bed joints of the mortar.

Micro-model is modelled using spatial eight node element SOLID45. Steel anchoring plates to insert the pre-stressing forces are modelled from the final element SOLID45. The pre-stressing bars weren´t modeled as a 1D element LINK8. This element was replaced with stress from pre-stressing bar [38].

Due to the behavior of masonry elements there is a limited field of loading almost linear curves up to the point of damage when can appear brittle fracture. For the mortar that is not true because its behavior is similar to concrete, which exhibits non-linear curves at low values of load in the compression zone. In contrast, in the tension zone development of cracks and thereby a reduction of the material properties occurs. In the numerical simulations is contemplated with the material linearity all input parameters apart from mortar [39], [40].

On the Fig. 6 and Fig. 7 are outputs from measurement which are compared with numerical simulation. In the numerical simulation is contemplated with the material non-linearity especially of mortar under anchor plate.

Tab. 2. Output values of deformation from measured and numerical model.

Loading process	Measured deformation		Deformation of numerical model	
	Direction A [mm]	Direction B [mm]	Direction A [mm]	Direction B [mm]
10%	0.140	0.060	0.135	0.128
20%	0.890	0.190	0.445	0.435
30%	1.080	0.450	0.761	0.744
40%	1.230	0.760	0.898	0.890
50%	1.450	0,950	1.123	1.111
60%	1,570	1.180	1.347	1.329
70%	1.670	1.220	1.566	1.562
80%	1.780	1.290	1.806	1.789

In the Table 1 are values of modulus of elasticity of mortar under anchor plate. For the values 40 to 80 % of the compressive strength of the masonry perpendicular to bed joints weren´t detected real values of modulus of elasticity. Therefore are used the same values as for 30 % of the compressive strength of the masonry perpendicular to bed joints. Modulus of elasticity of mortar wasn´t modified. When reducing modules of mortar under the measured values would not reflect reality and it would be no longer speculation. In the Table 2 is comparison between values measured and values from numerical simulation.

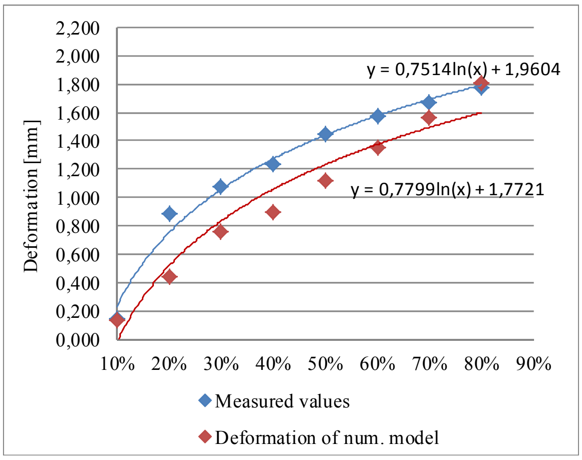

Fig. 6. Comparison in the direction A.

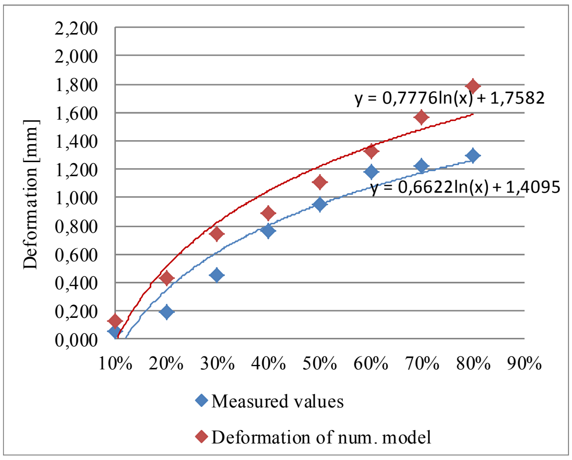

Fig. 7. Comparison in the direction B.

In the direction B were measured values of deformation lower. It is perhaps caused higher value of modulus of elasticity in the direction A. The differences may be caused by unevenness of the substrate. According to ČSN EN 1015-11 [41] is prescribed compaction of mortar in two layers. Each layer has to be compacted with 25 blows but this conditions can´t ensure on the experiment. Hereby occur different diagram of the sample taken of mortar and working diagram of mortar of real experiment.

The results show that by changing the material properties of mortar are resulting values to the numerical model of masonry significantly closer to the measured values. Considering how much masonry construction, which usually consisting of two elements with very different properties for modeling complex and compliance between simulation and measurement is quite challenging due to the many variables in the calculation, the results can be seen almost excellent. In this way, we can proceed further and change all values until we reach a perfect alignment of boundary conditions including variables in the calculation between numerical models and actually measured values. In practice, it is almost impossible to obtain all the necessary data for modeling. According to the above listed models sufficient obtain only appropriate number of samples and determine the necessary physical and material properties of brick and mortar, which serve as input variables in the calculation. The most important values are the strength of materials which are required for introducing the tensile force.

On the Fig. 9 and Fig. 10 are given vertical sections of masonry with anchor plate 150 x 150 x 10 mm and its measured values. In comparison with outputs of the numerical model on the Fig. 5 and Fig. 6 is deformation curve very similar in the place of pre-stressing bars and in the area of anchor plates.

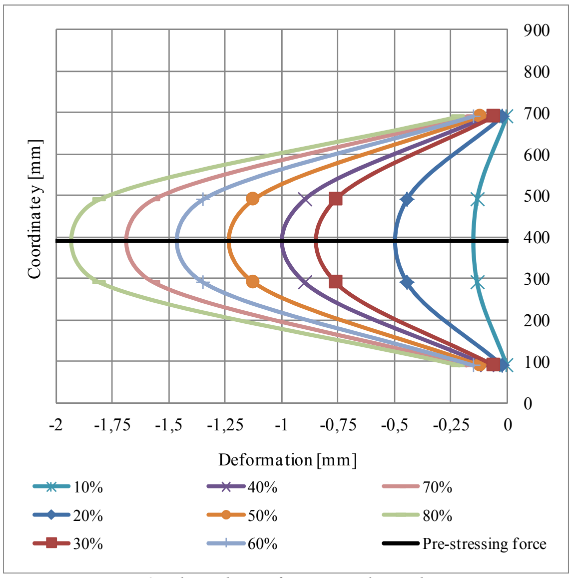

Fig. 8. The values of numerical simulation in the direction A (average).

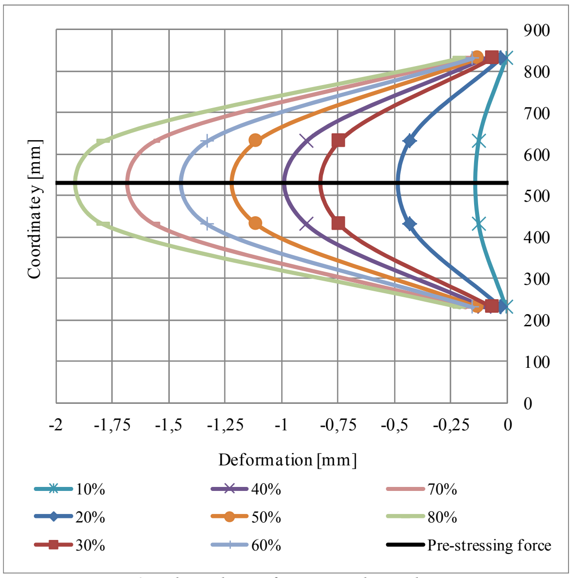

Fig. 9. The values of numerical simulation in the direction B (average).

Conclusion

The current conditions of the church - cracks in the walls and vault chords - have been caused by the undermining activities under the territory concerned. It is probable that the mining extraction will continue in the future, deteriorating thus the current condition of the building (increasing inclination and transverse torsion).

It follows from the analysis of past as well as expected mining impacts that the building under evaluation have been and will be influenced by the mining activities. It is evident from the technical/geological survey, site survey and masonry strength tests that the building can be protected against the undermining impacts by pre-stressing the reinforced foundations and head-pieces of walls under cornices, by providing the reinforcing rings, and injecting the cracks.

Using differently sized pre-stressing forces resulted, as expected, in linearly increasing stress and strain (displacement) in the structure. The model created from individual bricks and mortar showed higher local maximum values in the most exposed areas (termination of pre-stressed wire ropes). Stress and strain in these critical areas greatly affected mainly the nearest bricks and mortar, the other elements, however, are influenced only insignificantly.

Because the masonry is an inhomogeneous and anisotropic material consisting from masonry components about different physical and material properties is the creation of a suitable model that would expresses the actual material and physical properties of masonry difficult. For the experimental measurements other numerical models will be subsequently created, which will be fine-tuned during the experimental testing, so that these models, correspond with their properties, as much as possible to the actual behaviour of masonry with regard to the formation of cracks and brittle behaviour of bricks.

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