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EXPERIMENTAL MEASUREMENTS OF PRESTRESSED MASONRY WITH USING SLIDING JOINT

Abstract

Contribution deals with experimental measurements of deformations in the place exposed to local load caused by additional pre-stressing. The measurements are made at the masonry corner built in the laboratory equipment. The laboratory equipment was designed at Faculty of Civil Engineering VŠB-Technical University of Ostrava for measurement tri-axial stress-strain conditions in masonry. In this masonry corner two pre-stressing bars are placed. These bars are in different height and are anchored to the anchor plates, which transfer pre-stressing forces to the masonry. The specimen for laboratory testing is performed in the proportion to the reality of 1:1. In the bottom part masonry is inserted asphalt strip. It operates in the masonry like a sliding joint and reduces the shear stress at interface between concrete and masonry structures. The results are compared with the results of masonry without the use of sliding joints, including comment on the effect of sliding joints on the pre-stressing masonry structures.

Keywords

Experimental measurements, deformations, pre-stressing, masonry, sliding joint.

1 INTRODUCTION

The method of reducing of shear stress in the foundation joints using application rheological sliding joint is effective and its application in the praxis is very easy. The sliding joints are usually created using asphalt belt on the concrete layer or using mastic asphalt or plastic sheet. The current methods [1] of design of sliding joints are insufficient. Because the offer of materials is very large the calculations can´t be used. The correctness of design of rheological sliding joint is depends on the knowledge of mechanical response of asphalt belt during long-time active shear loading because the long-time deformations have influence on the structures.

Within of research of SGS grant on the VŠB-Technical University of Ostrava is performed to verify the suitability of combining remediation measures using pre-stressing of masonry and using of sliding joint into masonry.

For pre-stressing in the masonry is possible to use steel bars or cable. These steel elements are embed into additionally milled grooves which are external or internal wall face. The ends of steel elements are clamped to steel angles or special anchors. In these methods of reconstruction are necessary to observe the processes and technology tensioning. Important is appropriately choose pre-stressing systems, setting and embed of cables, preloading procedure and determining the size of the tensile forces in each cable.

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In the literature [2~10] is possible find recommended values of pre-stressing forces which was obtained on the base of experimental tests. In the literature [4, 5, 6] are the values which are in the plate of vertical of sectional horizontally pre-stressed of wall belt.

For experimental tests of masonry was built two testing samples. For easy references are identified MAS_1 a MAS_2 (masonry 1 and masonry 2). The both samples were built with using the same materials but the mortar was for each sample different and the strength of masonry was also different. Sample MAS_1 was built without using of asphalt belt and sample MAS_2 was built with asphalt belt in the heel of masonry. The asphalt belt in the heel of masonry simulated sliding joint in the masonry. For sliding joint was used oxidized asphalt belt, commercial name IPA V60 S35 [11,12].

The pre-stressing forces in the experimental measurements of deformation which is in this article are chosen safely with regard to the quality filling the joints with mortar as 10 – 50 % of masonry compression strength perpendicular to bed joints. These values are reached directly under anchor plate. The 50 % corresponds to approximately 12.5% of the voltage reached in the vertical of section of the wall horizontally tensioned belt. The purpose of this testing is not only the actual deformation measurement, but also monitoring the behavior of masonry at the local strain of gradually increasing preload.

The tests simulate the behavior of masonry reinforced pre-stressing cables at the moment of introduction of preload, it is the short-term tests.

2 PRINCIPLE OF MEASUREMENTS

2.1 Using material and its material characteristics

The equipment for testing tri-axial stress was steel structures about dimensions 900 x 900 x 1550 mm which was designed and built according to [13]. In this structure was built brick corner about height 870 mm. The thickness of wall was 440 mm. The surface of masonry was not plaster. Using material were bricks CP 290x140x65, P15 and lime mortar which was mixed with sand in the ratio 1:4.

The average strength of bricks according to [14] was 12.87 MPa for both testing samples. From this value is normalized average strength in the pressure f_b = 9.9 MPa. The average strength of mortar in the pressure according to [15] was $f_{m,1}$ = 0.77 MPa for sample MAS_1 and for sample MAS_2 was the value $f_{m,2}$ = 0.351 MPa.

Testing brick corner was considered as part of existing structures. For determining characteristic compressive strength of masonry was used standard [16] which refers to invalid standard for masonry [17]. The value of characteristic compressive strength of masonry was f_k = 1.663 MPa for sample MAS_1 and for sample MAS_2 was f_k = 1.366 MPa.

During of walling were placed into the masonry two pre-stressing bars. The bars were in the different heights. Each pre-stressing bar was identified according to direction (the direction A, the direction B), see Fig.1. The bar in the direction A was placed in the height 390 mm and in the direction B was placed in the height 530 mm. The pre-stressing bars were according to the manufacturer type of HPT 26 from steel 11 523 about diameter 26 mm, modulus of elasticity 185 ± 10 GPa. The bars were smooth without grooves or other surface finish. When all structure of brick corner was ready the upper part of structures was aligned mortared. Then on the structure was loaded the plate with thickness of 12 mm. On the pre-stressing bars were placed steel anchor plates. The masonry under the anchor plate was aligned mortared.

2.2 Loading of testing samples

The vertical loading was installed with using hydraulic press which was placed between the plate and I-profile, see Fig. 1. The both samples were imposed vertical load about value 0.1 MPa.

The pre-stressing force was installed into pre-stressing bars using hydraulic presses through anchor plates with dimensions 300 x 300 mm and the thickness of 10 mm or the thickness of 20 mm. The values of pre-stressing forces are in the Tab. 1.

The measure deformations were recorded using potentiometric sensors which were attached to laboratory equipment. In the each direction were attached eight sensors. In the direction A were identified from M21 to M28 and in the direction B were identified from M1 to M8, see Fig. 1.

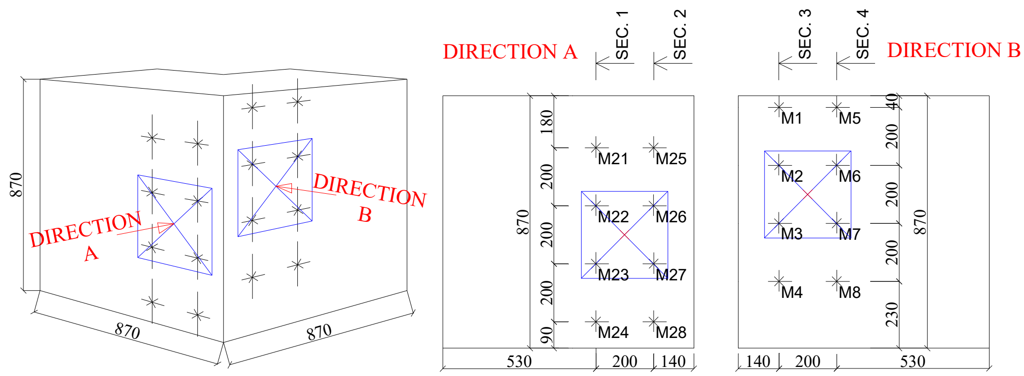

Fig. 1: Schema of the location of measurements sensors in the direction A and in the direction B

Fig. 2: Testing sample MAS_2, detail of sliding joint in the foot of masonry

The samples were loaded pre-stressing force about value from 10 % to 50 % of masonry compression strength perpendicular to bed joints the first in the direction B and then in the direction A. On the testing samples were performed only two measurements due to the elimination of errors during measurement.

In the Tab. 1 are input values of loading of masonry. In the first column are values of percentages. In the second and fourth column are value of stress in the anchor area and in the third and in the fifth column are values pre-stressing forces.

Tab. 1: Input values for pre-stressing of masonry, area of anchor plate $A = 0.09$ m^2

	MAS_1 ($f_k = 1.663MPa$)		**MAS_2** ($f_k = 1.366MPa$)	
	Stress [kPa]	Pre-stressing force [kN]	Stress [kPa]	Pre-stressing force [kN]
10 %	166.3	14.97	136.6	12.29
20 %	332.6	29.93	273.2	24.59
30 %	498.9	44.90	409.8	36.88
40 %	665.2	59.87	546.4	49.18
50 %	831.5	74.84	683.0	61.47

2.3 Results of measurements and its comparison

Waveforms resulting from deformation measurements can be seen in the following Fig. 3 and Fig. 10. The x-coordinate contains values of deformations with a negative sign induced by the pressure of anchoring plate on masonry. The resulting deformations are obtained by averaging of the measurements in vertical sections M21 ~ M24 and M25 ~ M28 in A direction and M1 ~ M4 and M5 ~ M8 in B direction. On the vertical axis there are elevation coordinates of the location of individual sensors. All sensors were placed on bricks or on anchoring plates, but not in the mortar joint. Horizontal line in the graph indicates the location of pre-stressing force.

The images of the both test samples MAS_1 and MAS_2 represent compression of anchor plates and their surroundings. Deformation graphs of anchor plates with different stiffness show that in the case of an anchor plates with a thickness of 20 mm there are higher deformation under the anchor plate and around than with using of the anchor plates with a thickness of 10 mm. The reason for this behavior of the anchor plate is higher flexural rigidity of the anchor sheets with a thickness of 20 mm. The flexural rigidity is dependent on the thickness of plate which enters into a relationship in the third power and the ratio of the rigidity of the two plates is 1:8.

It is evident from Fig. 3 to Fig. 6 the shape deformation of the masonry in both directions (at the place of pre-stressing bar) correspond the concentration of stress directly under the anchor plate. While the above the level and below the level of anchor plates are deformation almost negligible. Waveforms of deformation are approximately in the same intervals for each size of prestressing forces, especially in the direction of B.

To comparison the values of the resulting waveform of the test sample MAS_1 with plate 300 x 300 x 10 mm (Fig. 3 and Fig. 4) in both directions tensioned, is above the limit of stress corresponding to 50% of the strength of pressure perpendicular to the bed joint which operates directly under the anchor plates. While in the case of the sample with the plate 300 x 300 x 20 mm (Fig. 5 and Fig 6), there is a comparison of the values of the resulting deformation in both directions, even at the stress corresponding to 30% or more of the strength of pressure perpendicular to the bed joint operates directly under the anchor plates.

On the base results of testing sample MAS_1, it is possible to say, when are used anchor plate with higher flexural rigidity are deformation in the both direction almost the same (at lower of pre-stressing forces). Of course is the situation dependent on the high-rise location of anchor plate, the size of anchor plate and modulus of elasticity of mortar and of bricks.

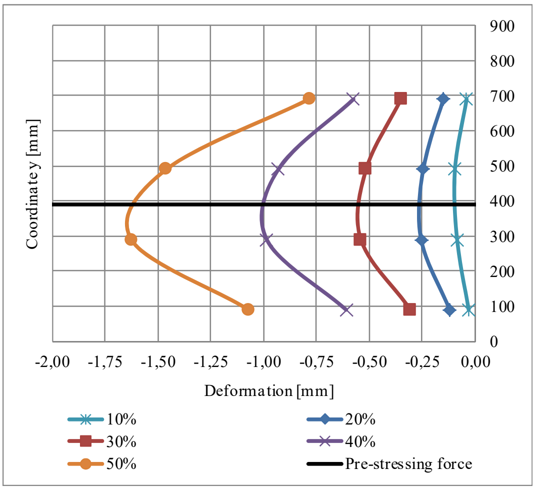

Fig. 3: Deformation MAS_1 in the direction A, anchor plate 300 x 300 x 10 mm

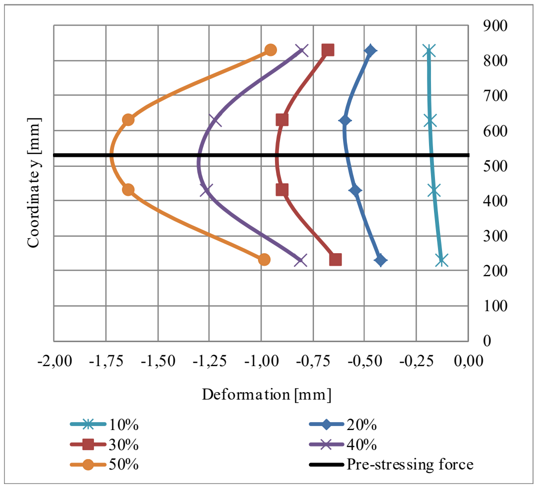

Fig. 4: Deformation MAS_1 in the direction B, anchor plate 300 x 300 x 10 mm

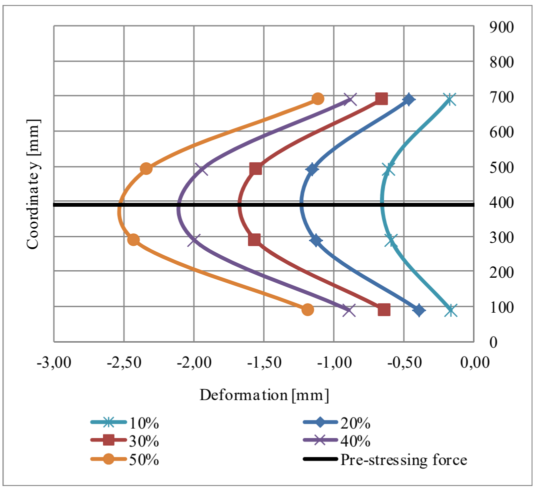

Fig. 5: Deformation MAS_1 in the direction A, anchor plate 300 x 300 x 20 mm

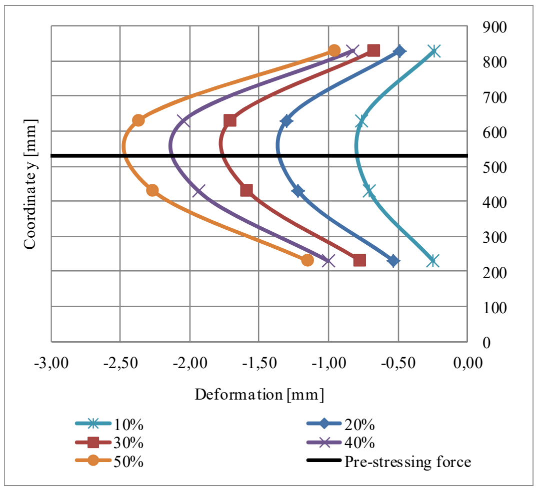

Fig. 6: Deformation MAS_1 in the direction B, anchor plate 300 x 300 x 20 mm

From Fig. 7 to Fig. 10 the shape deformation of the masonry in both directions (at the place of pre-stressing bar) correspond the concentration of stress directly under the anchor plate. While the above the level of anchor plates are deformation almost negligible at a position below the anchor plate and especially in the heel masonry can see the influence of the sliding joints on the resulting deformation.

Significant deformation is on the Fig. 7. There is used the anchor plate with a lower rigidity and positioning pre-stressing bars is 340 mm from the sliding joint. At this point it can be seen shift of masonry to asphalt belt simultaneously with increasing pre-stressing force (identical values in the heel and at the site of a pre-stressing force).

When anchor plate is used with larger thickness, see Fig. 9, is the deformation of masonry with sliding joint also significant. But there is not the shift of anchor plate together with sliding joint. These shifts are smaller than shifts anchor plate with smaller rigidity.

The influence of sliding joint of testing sample MAS_2 in the direction B is almost none. It is possible to say important for combining remediation measures using of pre-stressing masonry and using sliding joint is the high-rise location of the pre-stressing equipment and rigidity of anchor system.

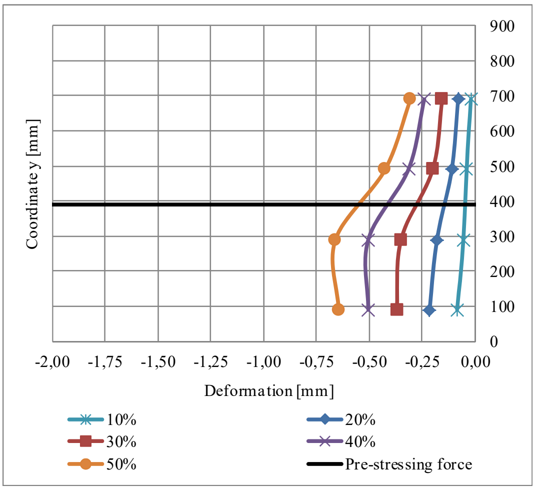

Fig. 7: Deformation MAS_2 in the direction A, anchor plate 300 x 300 x 10 mm

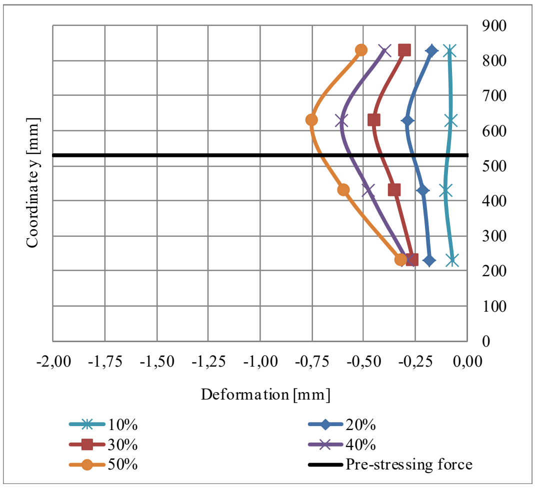

Fig. 8: Deformation MAS_2 in the direction B, anchor plate 300 x 300 x 10 mm

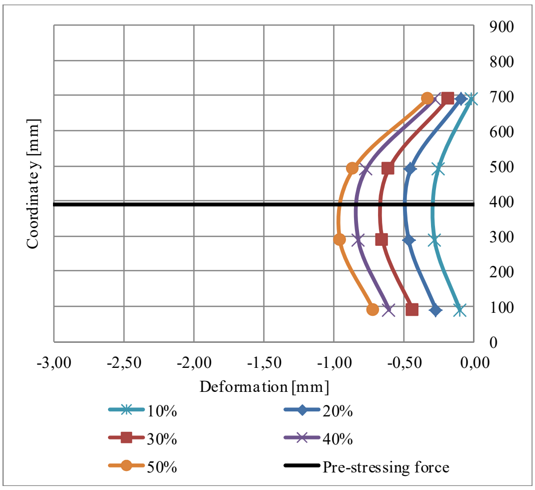

Fig. 9: Deformation MAS_2 in the direction A, anchor plate 300 x 300 x 20 mm

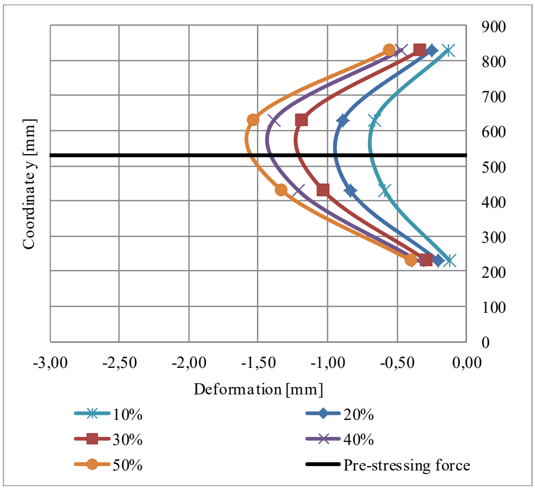

Fig. 10: Deformation MAS_2 in the direction B, anchor plate 300 x 300 x 20 mm

3 CONCLUSION

The article deals with laboratory measurements of pre-stressing of brick corner. Simulate cases strengthening and rehabilitation of masonry buildings.

The comparison of results deformation with using anchor plate with different thickness shows different compression in the place of anchor plates. The differences were by both samples. The differences were caused by different flexural rigidity of anchor plates. The rigidity of anchor plate with thickness of 20 mm is eight-time greater than rigidity of anchor plate with thickness of 10 mm.

For the verification of hypotheses is necessary to perform measurement of pre-stressing of masonry with using anchor plate with larger flexural rigidity than before. Then perform the comparison with already obtained values which are in this article.

The resulting deformations of masonry with using sliding joints are different. It depends which the material is used (for sliding joint) and where is located in the masonry, furthermore, on the high-rise location of the pre-stressing facility and rigidity of anchor system. Similarly measurement of sliding joint are addressed within of financial support from the Ministry of Industry and Trade, program TIP project number FR-TI2/746 – Rheological sliding joint with temperature controlled viscoelastic properties [18, 19, 20].

The aim of work should be to use software, on the base FEM for the design or assessment remediation of damaged masonry structures [21~29].

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