In situ tests for the assessment of seismic strengthening historic brick masonry walls with carbon fiber fabric

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SUMMARY:
Mechanical characteristics of existing brick masonry and the effectiveness of strengthening with carbon fiber fabric were investigated. Lateral resistance was tested in-situ on two wall specimens in a building from the nineteen thirties. Characteristics in compression were evaluated by in-situ flat jack tests and laboratory tests on wall specimens from the building. The results of compressive and tensile strength of brick masonry are comparable with the results of investigations on similar masonry in Slovenia. After lateral resistance tests of original specimens, they were strengthened with unidirectional woven carbon fiber fabric. Crack pattern and failure mechanism changed with regards to original masonry. Lateral resistance was increased by 42 and 60%. No reasonable difference in lateral resistance was observed between the case where fiber fabric was applied equally on both sides and on one side incompletelly. Significant increase of stiffness at maximum resistance was observed in both cases, amounting to 3.3-times.

Keywords: Brick masonry, historic, strengthening, carbon fiber fabric

1. INTRODUCTION

Old brick-masonry buildings form a major part of many cities in Europe, forming either their historical nuclei or standing individually. Since these buildings represent a valuable architectural cultural heritage, they need to be preserved for future generations. Although the structural typology of these buildings varies from region to region, their common drawback is that they have not been designed to resist seismic loads. They have been built using materials and systems which can resist the compression caused by the gravity loads, but cannot resist the bending and shear loads which result from the ground motion due to earthquakes.

Masonry walls are usually built out of solid bricks, in pure lime mortar or mortar consisting of sand, lime and a little cement, which means that such masonry has a relatively low strength. The typical thickness of such structural walls varies, depending on the height of the building. Partition walls are less thick, but built like structural walls. In the case of higher brick masonry buildings a step, decreasing the thickness of structural walls at every or every second floor, is very common. It provides not only a very similar level of stress in the structural walls due to gravity loads, but also an advantageous distribution of seismic forces and consequently lower inertial forces in the structure. The floor structures usually consist of brick vaults over cellars, ground floors and corridors, and wooden floors over the rooms in the upper storeys. The main disadvantage of wooden floors is that they are insufficiently rigid, and inadequately anchored to the walls. On the other hand, they are of quite limited weight and consequently do not induce high seismic loads.

A large number of such buildings also exist in Slovenia (Lutman 2010). Many of them were built in the period before the Ljubljana region was hit, in 1895, by a strong earthquake, raising general awareness of seismic risk. Whereas previous building practice had been more or less based upon experience, after this earthquake a seismic code was prepared for the first time in order to prescribe measures for the provision of adequate seismic safety. It was a code for the earthquake-resistant construction of buildings, a very prescriptive-type of code, which was based mainly upon observations of how structures suffer damage during earthquakes. The quantity of load-bearing walls in each of the floor-plan directions is approximately the same, flat iron tie-bars are installed at the height of the floor structures, and the floor structures themselves are anchored into the walls.

Many experimental and analytical investigations have been carried out over the last decades (Zimmermann 2010, Tomaževič 2011) in order to obtain adequate information about the seismic behaviour of masonry buildings. The most valuable data of old masonry are obtained by in-situ tests or at least laboratory tests on masonry specimens from existing walls. With these types of tests, the actual old masonry is investigated directly, without searching for the best replacement for it. It is namely very difficult to reproduce the existing masonry in the laboratory, although using mortar with adequate chemical and mechanical characteristics. While getting very close to the original quality of old bricks, the biggest problem is usually the preparation of mortars and appropriate method of the construction. The advantage of in-situ tests are also realistic boundary conditions of the specimen and its fixing to the surrounding masonry. For these reasons, we are glad of any opportunity to perform in situ investigations. Since this is a destructive test method, the owners of buildings, of course, do not agree with such investigation in their building, unless they intend to demolish. Such an opportunity has appeared recently.

2. DESCRIPTION OF THE BUILDING

The building was built in 1935 in its current narrow center of Ljubljana and is a typical building of that time. Since the building on its site, has become dysfunctional, it was demolished in 2010. Before that, we agreed with the owner to carry out investigations. One part of investigations on the masonry was performed in the building and the other part in the laboratory on samples of bricks and small walls.

Figure 1. Typical multistorey residential building from the period between WWI and WWII in Ljubljana, where the in-situ tests were carried out

The building layout was slightly irregular rectangular shape with a flat road façade and segmented courtyard façade. The building was 20.50 m long, 11.27 m wide and extended for the 2.00 m at three points to the rear side. The basement was not completely below the ground level and the ground floor was slightly above the ground level. The building had been basically built with two floors and the attic was utilized only at the side of the courtyard. Later, the building was raised for one floor. Storey heights amounted to 3.05 m in the basement and 3.70 m in other floors. These storey heights were typical for those days, slightly higher than in buildings of today.

While the foundations and basement walls were constructed out of concrete, all other walls were built of solid bricks of normal size. The brick walls had been constructed with a lime mortar with a small amount of cement. There were reinforced-concrete slabs above the cellar and timber floors above ground floor and upper floors, respectively. Timber joists were laid in the transverse direction of the building and mostly loaded longitudinal load-bearing walls. These were arranged in two lines of the road façade and courtyard façade, and one inner line. Since they were loaded with floor structures, the longitudinal walls were thicker than the transverse ones. While the internal longitudinal walls were 51 cm thick in all floors, the outer longitudinal walls were 51 cm thick in the ground floor and 38 cm in the upper floors. The main transverse walls were 38 and 25 cm thick. Transverse partition walls, which were also built of solid bricks, were 12 cm thick. Typical view to the road façade and typical floor plan are given in Figs. 1 and 2, respectively.

3. EXPERIMENTALLY OBTAINED MECHANICAL PROPERTIES OF ORIGINAL MASONRY

After the building had been inspected visually and the load-bearing system had been found out, two typical walls in the ground floor were chosen to carry out the in-situ lateral resistance tests in order to determine the tensile strength of original masonry. In addition to these in-situ tests, two wall specimens had been cut out of walls, brought to laboratory where the compression tests were performed. Five couples of solid bricks had been also taken out from the masonry and the compressive strength of bricks was obtained in laboratory. The positions of tested walls and places of samples taken out of masonry are shown in Fig. 2.

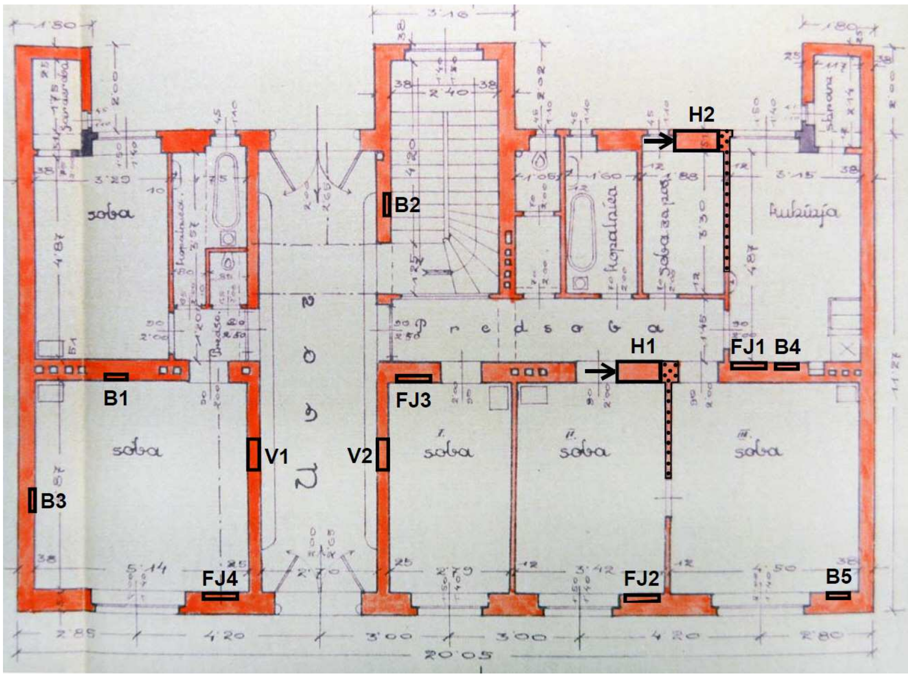

Figure 2. Plan of ground floor with positions of in-situ lateral resistance tests (H1 and H2), flat-jack tests (FJ1 to FJ4) and places of the extraction of brick specimens (B1 to B5) and small walls for laboratory compression tests (V1 and V2)

3.1. Compressive strength of bricks

The nominal dimensions of solid bricks used in Slovenia after the First World War were: 25 x 12 x 6.5 cm (length x width x height), but the actual lengths varied between 24.0 and 25.0 cm and their widths varied between 12.1 and 12.4 cm. The compressive strength was determined on five brick couplet specimens. The average value of compressive strength f_B = 11.3 MPa and the coefficient of variation 0.32 were obtained out of the results.

3.2. Compressive strength of masonry

Compressive strength of original brick masonry was obtained from the laboratory tests on two wall specimens which had been taken out of the building and transported to laboratory. Transversal walls in the ground floor, V1 and V2 in Fig. 2, appeared as most approachable and convenient for taking specimens out of the surrounding masonry. Each specimen was separated from the masonry by cutting it at all four edges by means of a saw, put between two rigid steel plates, tied up and transported to laboratory. The wall specimens for the compression tests were 0.65 m long, 0.25 m thick and 0.83 m high. They were tested in a 5000 kN testing machine, by subjecting them to the loading procedure according to EN 1052-1. The vertical load was increased steadily without unloading until failure was reached after 15-30 minutes from the commencement of loading. The specimens were instrumented with a load cell and displacement transducers (LVDT's) in order to monitor the load - deformation relationship. First cracks developed at approximately one half of compressive strength and propagated until the maximum vertical load was achieved. Crushing and falling-off of separated layers followed up to the failure of the specimen (Fig. 3 on the right).

The experimentally obtained compressive load - displacement relationship is presented in Fig. 3 on the left, in the form of a stress-strain diagram. Strain values in this diagram were evaluated from the front and back measurements. The modulus of elasticity was derived from at one third of the compressive strength. The test results are summarized in Table 3.1.

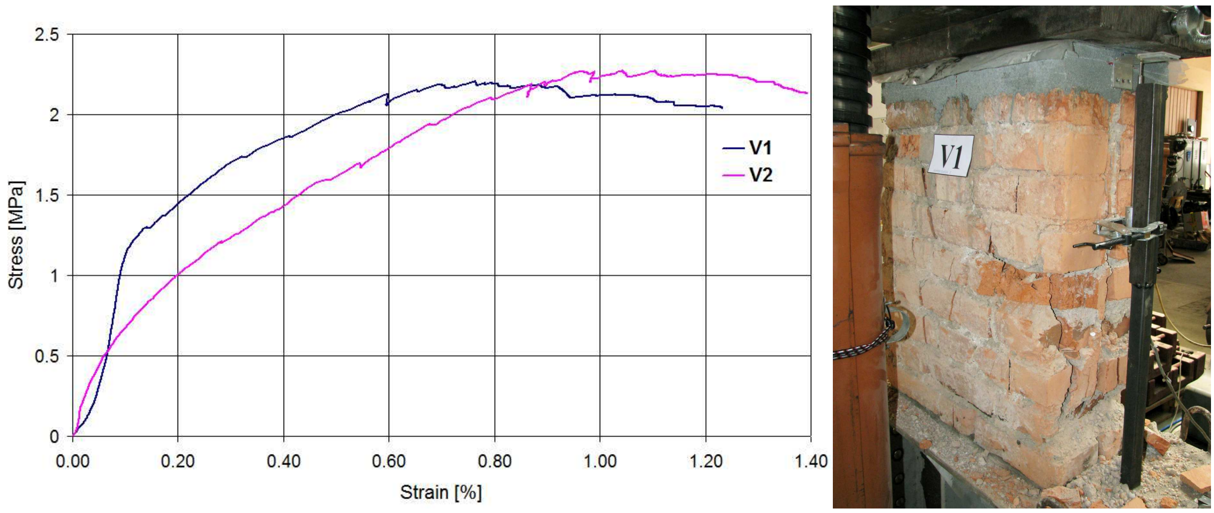

Figure 3. Relationship between the vertical load and strain of specimens during compression tests (left) and typical compression failure mode of specimen V1 (right)

Table 3.1. Compressive strength and modulus of elasticity obtained by compression tests in the laboratory

	V1	V2
Compressive strength f_c	2.21 MPa	2.27 MPa
Strain at maximum attained vertical load	0.76%	1.04%
Modulus of elasticity E	951 MPa	624 MPa

In addition to laboratory tests, compressive strength of masonry walls was tested in-situ by means of double flat-jack tests at positions FJ1, FJ2, FJ3 and FJ4 (Fig. 2). The average value of compressive strength f_c = 2.63 MPa and the coefficient of variation 0.18 were obtained out of the results. The results for modulus of elasticity resulted in the average value E = 3695 MPa and the coefficient of variation 0.49. The experimentally obtained compressive stress-strain relationship and the view of flat-jack test at position FJ3 are presented in Fig. 4.

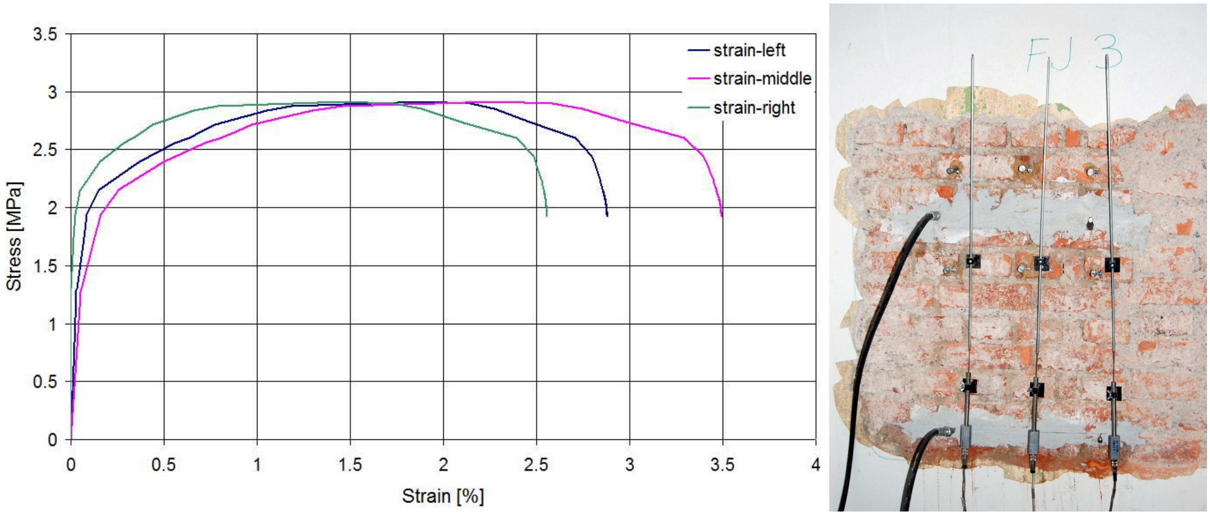

Figure 4. Double flat-jack test at the position FJ3: relationships between stress and strain (left) and arrangement of double flat-jack and instrumentation with LVDT's (right)

3.3. Lateral resistance of original masonry

Taken into account the type of floor structures and the path of the vertical load, only longitudinal walls of the building were sufficiently compressed by the weight of the upper structure and thus suitable for in-situ lateral resistance tests. Two locations were selected in the ground floor, one in the central wall (H1 in Fig. 2) and the other in the outer wall (H2 in Fig. 2).

Individual specimen was separated from the surrounding masonry by cutting the wall at both vertical sides by means of a saw. The dimensions of specimens had been previously determined on the basis of the estimated vertical load in selected positions, the boundary conditions and the required failure mechanism. However, in order to attain the adequate data for the evaluation of the tensile strength of masonry (Turnšek, Čačovič 1971), the shear failure mechanism was needed to develop. The dimensions of specimens are given in Table 3.2. Both specimens were tested as fixed-ended, by applying the lateral load at the middle of their height.

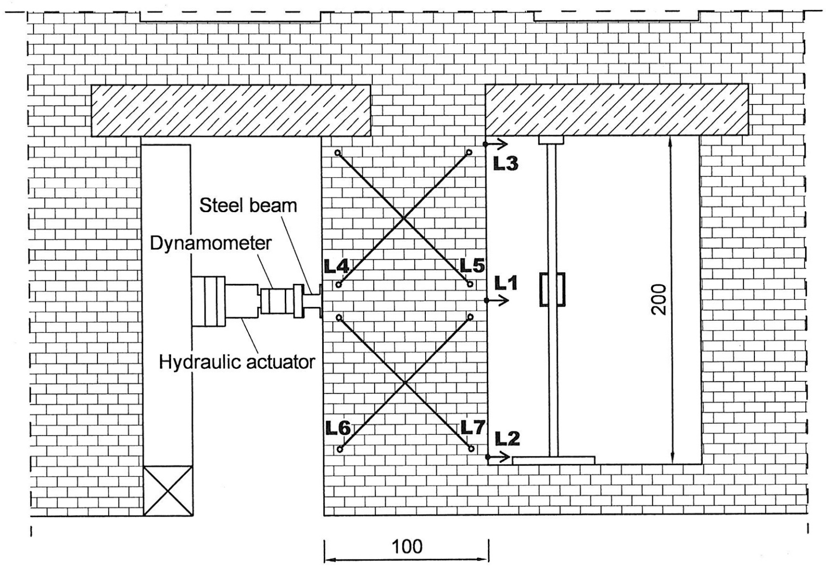

Figure 5. In-situ lateral resistance testing set-up

The lateral load was applied by means of a 500 kN capacity hydraulic jack and a system of steel beams and connecting rods. The specimen was instrumented with LVDT's to measure lateral displacements at the mid-height of the specimen and at both ends, as well as the strains in diagonal sections of both halves of the specimen (Fig. 5). The lateral displacement, imposed at the mid-height of the specimen, was monotonically increased, with unloading at each amplitude. The procedure was repeated with step-wise increasing amplitudes of lateral displacement, also beyond the resistance of the specimen.

Typical views and crack patterns of specimens H1 and H2 at maximum resistance are shown in Fig. 6, while the relationships between lateral load and displacement, obtained by testing are shown in Fig. 7. Due to some uncertainties in the measurements of lateral displacement in the initial stages, the lateral displacement was evaluated from the measured values of diagonal strains. However, the amplitudes of the derived lateral displacement are of the same scale as the measured ones.

Figure 6. Arrangement of hydraulic jack and instrumentation of specimen H1 (left) and typical crack pattern at maximum resistance on the rear surface of specimen H1 (middle) and specimen H2 (right)

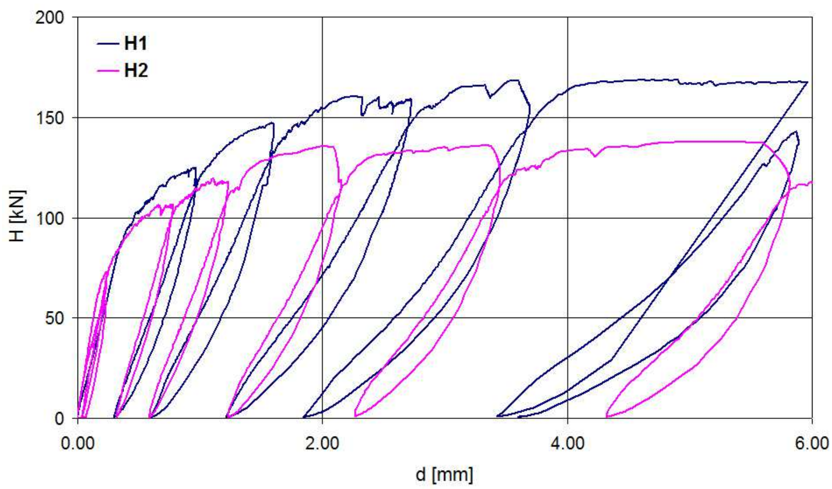

Figure 7. Relationships between lateral load and displacement of specimens H1 and H2

Since both specimens failed in shear, the tensile strength of the masonry f_t as the principal tensile stress in the wall at the attained maximum resistance (Turnšek and Čačovič, 1971) was derived:

$$f_t = \sqrt{\left(\frac{\sigma_o}{2}\right)^2 + \left(b\tau_{H\max}\right)^2} - \left(\frac{\sigma_o}{2}\right), \tag{3.1}$$

where σ_o is the average compressive stress in the horizontal cross-section of the wall due to vertical load, τ_{Hmax} is the average shear stress in the horizontal cross-section of the wall at the attained maximum resistance H_{max}, and b is the shear stress distribution coefficient, which depends on height/length ratio and vertical/lateral load ratio at the attained maximum resistance. Compressive stresses in both walls were assessed after the composition of floor and roof structures of the building had been inspected. By assuming the value b = 1.1 the results of in-situ testing are summarized in Table 3.2.

Table 3.2. Dimensions of tested walls and test results

	H1	H2
Dimensions (Length / Thickness / Height)	1.03 m / 0.52 m / 2.01 m	1.00 m / 0.52 m / 2.00 m
Average compressive stress σ_o	0.61 MPa	0.44 MPa
Maximum resistance H_{max}	169 kN	138 kN
Displacement at maximum resistance d_{Hmax}	4.86 mm	5.01 mm
Tensile strength of masonry f_t	0.157 MPa	0.144 MPa

4. STRENGTHENING OF MASONRY WITH UNIDIRECTIONAL WOVEN CARBON FIBER FABRIC

4.1. Description of strengthening

In order to evaluate the effectiveness of strengthening of historic brick masonry, both wall specimens, previously tested with lateral load in original condition, were strengthened and tested afterwards in-situ. They were strengthened with unidirectional woven carbon fiber fabric in epoxy resin. First, the surfaces of the wall, where the strips of fabric were intended to be applied, were levelled with thin layer mortar containing epoxy resin and fillers, without previous repair of cracks.

Figure 8. The application of carbon fiber fabric in epoxy resin: specimen H1w after placing the inclined strips on one side (left), complete specimen H1w with inclined strips and horizontal strips, equally on both sides (middle) and the outer side of specimen H2w only with horizontal strips (right)

In the case of specimen H1w, inclined strips were placed first, separately to both sides of the wall. These strips were applied in diagonal direction between the level of the applied load and the corners of the specimen, forming a cross-like pattern in the upper and bottom halves of the specimen (Fig. 8 left).

Additionally, horizontal strips of the fabric were applied at the top, middle and bottom of the specimen, which fully wrapped the wall. Strengthened wall H1w is shown in Fig. 8 middle.

The second specimen (H2w) was strengthened using the same materials. The inclined strips were applied only on one side of specimen, while the three horizontal strips of fabric were applied around the wall as in the case of specimen H1w. The side without inclined strips is shown in Fig. 8 right.

4.2. Lateral resistance of strengthened masonry

The strengthened specimens were tested with the same procedure as the original ones. The experimentally obtained lateral load - displacement relationships for both specimens are presented in Fig. 11 and typical values are summarized in Table 4.1. By analogy with original specimens, the lateral displacement was evaluated from the measured values of diagonal strains.

The crack propagation of strengthened specimens was different from crack propagation of original specimens. In the case of specimen H1w horizontal tensile cracks developed in the bed joints of those areas that were not covered with fabric. In contact with the fabric, cracks either propagated into the fabric (Fig. 9 left) or changed the direction and continued along the edge of the fabric (Fig. 9 right). In addition, diagonal cracks developed in the uncovered triangle zones. Only at the very end of the test, compressed part of masonry bellow the point where horizontal load was applied started to crush (Fig. 9 right – the bottom half of the wall).

Figure 9. Specimen H1w: the development of first cracks (left) and the final stage of the test (right)

Similar crack propagation was observed on the inner surface of specimen H2w (Fig. 10 left), which was covered with diagonal strips of fabric. Horizontal cracks in the bed joints developed in those tensile areas which were not covered with the fabric. All such cracks propagated through fabric. In the compressed zones vertical cracks developed on the uncovered side surfaces above and bellow the point where lateral load was imposed. At the end of the test, one crack developed within one of diagonal strips along its fibers.

Different crack pattern was observed on the outer surface of specimen H2w, where only horizontal strips were applied (Fig. 10 right). It was more similar to shear mechanism, with diagonally oriented cracks, limited to the area between horizontal strips of fabric and therefore less steep in comparison with shear cracks of the original specimen.

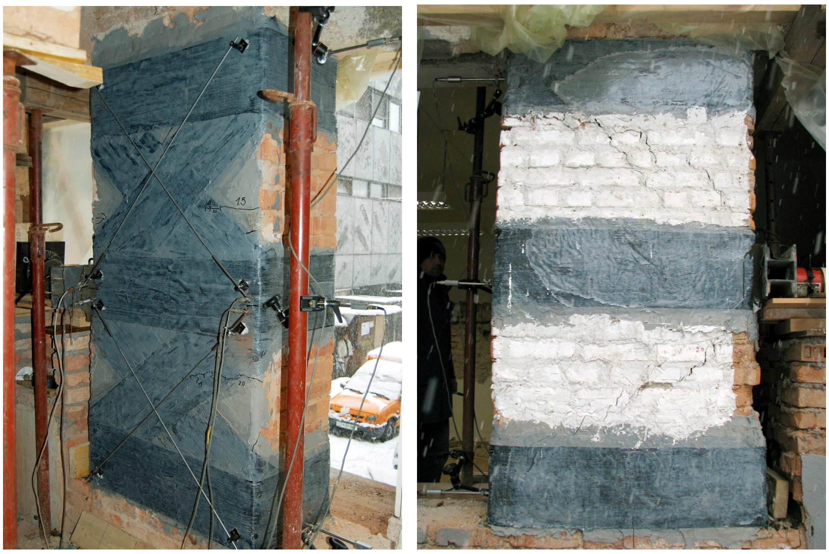

Figure 10. Specimen H2w: distribution of cracks on the inner surface (left) and on the outer surface (right)

In late stages of both specimens (H1w and H2w) two horizontal tension cracks occurred at the top and at the bottom of the wall, apparently due to the fact that the fabric was mounted on the wall only within its height, without being anchored to the lintel above neither to the masonry below the specimen. These cracks allowed the wall that it began to rotate. Besides, cracks in compression toe extended to the brick base of the specimen. The remaining lateral resistance in these stages was already smaller than the maximum resistance.

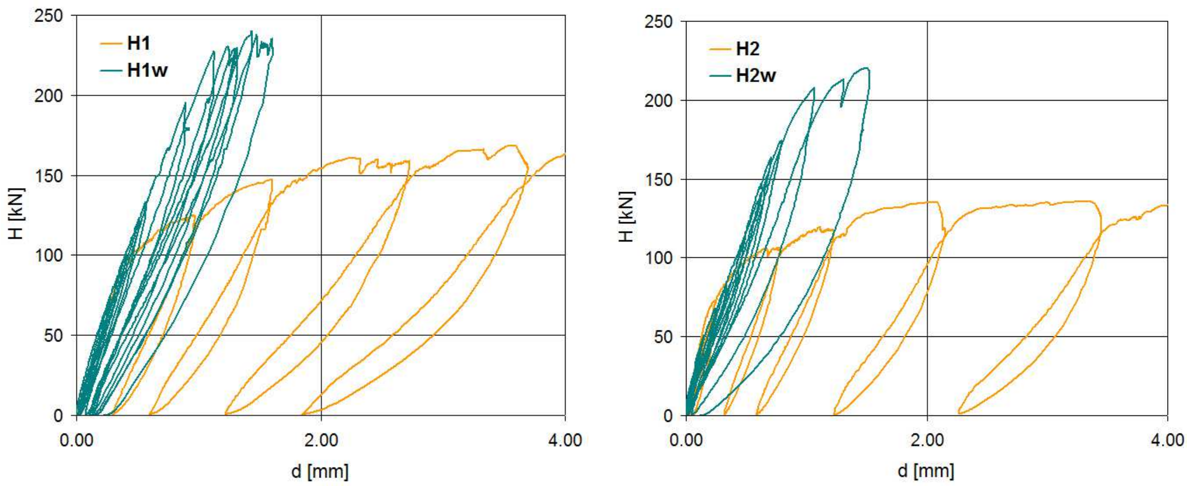

Figure 11. The influence of strengthening of specimens on the relationships between lateral load and displacement

Table 4.1. Test results of lateral resistance tests of strengthened specimens

	H1w	H2w
Maximum resistance H_{max}	240 kN	220 kN
Displacement at maximum resistance d_{Hmax}	1.43 mm	1.50 mm
Effect of strengthening $H_{max\text{-}wrap}$ / $H_{max\text{-}orig}$	1.42	1.60
Effect of strengthening $d_{Hmax\text{-}wrap}$ / $d_{Hmax\text{-}orig}$	0.30	0.30

As expected, the performed strengthening significantly increased the shear resistance of specimens (Table 4.1). The increase of lateral resistance was, contrary to our expectations, even higher in the case of H2w than in the case of H1w. The performed strengthening of masonry also significantly increased the stiffness at maximum resistance.

5. CONCLUSIONS

Mechanical characteristics of existing brick masonry were evaluated on the basis of in-situ tests in a building from the nineteen thirties and additional laboratory tests on specimens of masonry, taken from the same building. In-situ lateral resistance tests were carried out on two wall specimens. Since both specimens failed in shear, without cracks in the surrounding masonry, the average value of tensile strength of original masonry f_t = 0.151 MPa was evaluated from test results.

Compressive strength and modulus of elasticity were determined by flat jack tests in the building and by laboratory tests on smaller wall specimens, cut out from the building walls and brought to the laboratory. While the average values of compressive strength from laboratory tests (f_c = 2.24 MPa) and from in-situ tests (f_c = 2.44 MPa) are very close, significant difference in average values of modulus of elasticity (E = 788 MPa and E = 3695 MPa, respectively) was observed. Nevertheless, all values of mechanical characteristics and relationships between them are very comparable with the results of investigations on similar masonry in Slovenia (Tomaževič, 1999). The differences between the values of modulus of elasticity would be advisable to explore with regard to boundary conditions of particular type of test.

The effectiveness of strengthening of wall specimens with unidirectional woven carbon fiber fabric was investigated with regard to damage propagation and failure mechanisms, as well with regard to lateral resistance and stiffness. The maximum resistance of the first specimen with diagonal strips of the fabric equally on both sides and three wraparound horizontal strips was improved by 42%. In the case of specimen H2w, with diagonal strips only on one side, the increase of lateral resistance was, against our expectations, even 60%. It is unlikely to attribute this discrepancy to the fact that this specimen was loaded with lower vertical load. The deformation capacity at maximum resistance was reduced by 3.33 times in both cases. Until the maximum capacity has been reached, cracks developed on the surfaces that were not covered by the fabric. In the case of specimen H2w, with diagonal strips only on one side, an interesting progression of cracks was observed. The crack patterns of the different sides were different. This is likely to be enabled by the nature of masonry bonds and flexibility of mortar. In final stages of tests of both specimens, it proved to be inadequate that fiber fabric was applied only within the wall height. It enabled the formation of horizontal cracks at the edges of walls and the rotation of walls. These observations showed that carbon fiber fabric should be anchored to the surrounding masonry.

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