Combined in-situ tests for the assessment of historic masonry structures in seismic regions

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ABSTRACT: The knowledge of the structural behaviour of existing masonry requires a multilevel approach, with proper application of diagnostics and assessment methodologies. The paper presents the results of some recent in-situ tests carried out in collaboration by the Slovenian National Building and Civil Engineering Institute, the Polytechnic of Milan and the University of Padova, on the walls of the Pšence Castle, located in the Eastern part of Slovenia. In order to calibrate different test methodologies a combination of DT (direct shear tests on a double-panel), MDT (single and double flat jacks) and NDT (georadar, direct sonic transmission and tomography) were performed. The aim of the study was to define the morphological, compositional and structural characteristic by direct inspection and MDT, to qualify the mechanical properties of the masonry by MDT and DT, to compare results obtained by the different level of obtrusiveness procedures, and, finally on the basis of these results, to evaluate load bearing capacity of the Castle as well as its seismic resistance by means of push-over analysis.

1 INTRODUCTION

The case study presented herein, represents an example of NDT, MDT and DT based approach in numerical modeling of historic masonry structures. Following the extensive in-situ testing program carried out at Pšence castle (Slovenia) from 10th till 20th of July 2003, within the framework of the 5th European Framework project ONSITEFORMASONRY, the seismic analysis of the Western part of Pšence castle by means of push-over method has been evaluated.

Investigation of the Western part of Pšence castle was conducted to determine as-built and current conditions as well as mechanical properties of historic masonry. It was done through application of different techniques: in-situ shear test, radar, impact-echo, flat-jack, sonic test, videoboroscopy, microseismic waves, coring and sampling as well as the laboratory tests on mortar and stone samples. Partners involved in the testing campaign of the Western part of Pšence castle were: Bundesanstalt für Materialforschung und -prüfung (Germany), Politecnico di Milano (Italy), Slovenian National Building and Civil Engineering Institute (Slovenia), University of Padua and University of Pisa (Italy). In this paper some of the results of importance for the structural analysis of the building are presented and commented.

Following the results of DT and MDT tests, an attempt to calibrate some of the NDT tests have been also made. The results of numerical analysis provided valuable information of load-bearing capacity of the structure, possible causes for the observed crack pattern and validation of some of the results gained through on-site investigations.

2 DIAGNOSIS STRATEGIES

A correct intervention on a historic structure should start from an accurate diagnosis of the building in order to minimize the interferences of the intervention with the historicity of the architectural document. Furthermore, the study of the building rules and structural details, such as the wall sections, has a great importance in the mechanic behaviour of the structure.

The structural performance of a masonry can be understood provided the following factors are known: its geometry; the characteristics of its masonry texture,
single or multiple leaf walls, connection between the leaves, joints empty or filled with mortar, physical, chemical and mechanical characteristics of the components (stones, mortar); the characteristics of masonry as a composite material. In order to fulfill these needs an experimental on site investigation is required and recommended also by Codes of Standards in several countries.

NDT can be helpful in finding hidden characteristics, such as internal voids and flaws and characteristics of the wall section (Schuller et al. 1994, Valle et al. 1998, da Porto et al. 2003), which cannot be known otherwise than through destructive tests. Up to now most of the ND procedure can give only qualitative results. The application to masonry of NDT, although advanced, can be frustrating due to several factors, like the different masonry typologies and materials, the high inhomogeneity of the materials, the interpretation of the results of each single technique but also the harmonization of the results. Furthermore, most of the NDT’s come from other research field and need a specific calibration.

The solution of very difficult problems cannot be reached with a single investigation technique, but with the complementary use of different techniques (Binda et al. 2003a,b). Therefore the designer is asked to interpret the results and use them at least as comparative values between different parts of the same masonry structure or by using different ND techniques. To this purpose it is important the production of guidelines for the correct application of investigation techniques to diagnosis problems of different classes of masonries (Binda et al. 2000).

Often MDT or slightly destructive techniques should be applied in strategic points of the structure in order to solve the more difficult problems of hidden situations.

However, an ultimate approach represents DT procedure for the evaluation of compressive or shear (tensile) strength, as well as stiffness parameters and ductility of as-built historic masonry. This approach is not completely destructive, however the tested walls should be repaired after applied DT tests. This approach is inevitable for precise and accurate assessment of the structural behaviour of the building.

The diagnosis process should be based on an accurate survey, which should document the current state of the building. A preliminary in-situ survey is useful in order to provide details on the geometry of the structure and in order to identify the points where more accurate observations have to be concentrated. Following this survey a more refined investigation has to be carried out, identifying irregularities (vertical deviations, rotations, etc.). In the meantime the historical evolution of the structure has to be known in order to explain the signs of damage detected on the building. The crack pattern should be classified and accurately documented by pictures and on the geometrical survey. The definition of the structural model can be carried out only on the base of the geometrical survey but also of the crack pattern.

3 NUMERICAL STRATEGIES

In general, the seismic analysis of masonry structures can be done by using lumped parameter models (LPM), structural element model (SEM) or finite element models (FEM). Since for ordinary masonry buildings, the first vibration mode shape is the predominant one, there is usually no need for sophisticated non-linear dynamic and the LPM models are quite rare.

SEM approximates the actual structural geometry more accurately by describing individual structural elements such as piers and walls. In the case of single or multistory buildings due to their regularity and simplicity an equivalent static analysis in two orthogonal directions by using SEM can provide reliable information regarding the seismic safety under expected seismic loads. Nevertheless, since seismic loading can exercise the structural system to and beyond its maximum resistance capacity, the SEM models usually have to be used with static non-linear analysis. In that case a step-by-step procedure is followed, using decreased stiffness values under increasing lateral loads. Nonlinear element behaviour is prescribed in the form of nonlinear lateral deformation-resistance relationships, depending on the boundary conditions and failure mode of masonry elements. Usually the bi-linear or tri-linear behaviour of SE is considered. The storey resistance envelope is calculated by step-wise drifting of the storey for small values. The SE’s are deformed equally (due to the rigidity of floor structure) and internal forces are induced according to the assumed shape of resistance envelope of each SE. In the case of torsional effects (due to relatively displacement of the mass centre to the centre of stiffness of the storey) the displacements of individual SE are modified.

Masonry is a composite, heterogeneous, non-linear structural material. As with other composite materials, also with masonry the mechanical properties are conditioned with the properties of composite elements, their volume ratio and the properties of bond between the units and the layers of mortar. Moreover, the properties and behaviour of masonry is strongly affected by the orientation of the main principle stresses towards the bed joints. Following the aforementioned, the main strategies for application of FEM can be adopted for the masonry as follows:

- simplified micro-modelling – usable for small elements and shear wall with openings, where
expanded units are represented by continuum elements whereas the behaviour of the mortar joints and unit-mortar interface is represented by discontinuous elements.

- homogenization – is aimed to solve the problem of modelling of large masonry structures, by treating masonry as a homogeneous material. Mechanical properties of masonry are predicted from the properties of its constituents, i.e. units and mortars.

- macro-modelling – for modelling whole structures where masonry is regarded as an anisotropic composite material (constitutive models).

Each of those FEM strategies has advantages and disadvantages. However, when analyzing old historical building, where a limited parameters from different NDT/MDT techniques can be obtained, the effective FEM analysis by applying sophisticated constitutive material laws could be questionable.

For the Western part of Pišče Castle, SEM model with applied push-over analysis were adopted for the verification of the seismic resistance. Main input parameters were determined from the results of in-situ shear tests as well as the results from single and double flat-jack tests.

4 CASE STUDY – WESTERN PART OF PIŠECE CASTLE

The castle of Pišče was built at the first half of the 13th century as defence fortification against the Hungarian attacks by archbishop of Salzburg. First records about the castle dated from 13th century (1268 AD). Till 14th century it belonged to knights of Pišče. By the end of 16th century (1595 AD) castle was sold to Inočent Moscon. Since that time till the end of Second World War (1945), it belonged to Moscon family. After the Second World War, the castle became property of state and was used as social apartments. From the year 1998 the castle stands empty.

The oldest part of the castle is Romanic tower build in first half of 13th century with two and half meter thick walls, peripheral castle walls and Romanic chapel. The important renovations were made in year 1568 and the round tower was added on NW (North-West) side of the castle. In the baroque time some architectural details such as small tower over the chapel were added. In XVIII century considerable changes were made on the tower of Pišče Castle and its height has been reduced by removing the top floor of the tower. The last renovations were made in 1867 with neoromantic and neogothic decorations with partial reconstructions of existing walls and by closing some of the openings in the castle.

Figure 1. Western part of Pišče Castle.

Figure 2. History of building of Pišče Castle.

5 LAYOUT OF THE WESTERN PART AND MORPHOLOGY OF THE WALLS

From the structural point of view the Castle represent classical example of masonry structures for Slovenian region. It consists of main defence tower, attached buildings around it, a chapel and defence wall. Structural elements that determine the load-bearing capacity of the castle are mainly solid walls.

The Western part of the castle (Fig. 3), was built later in renaissance period and it is partly attached to the main tower. It is to be expected that its seismic behaviour would be strongly affected by the presence of stiff and strong tower in its vicinity and thus two directions of seismic analysis were determined as it is presented in Figure 3.

6 ON SITE TESTS

The walls of the Western part of Pišče castle appeared very inhomogeneous with signs of past interventions. On-site tests were carried at Positions 4a & b (Fig. 3) in order to control peculiar problems and situations.
The first inspection was made by radar profiles at Position 4b that clearly showed a position where the signal could not pass through the wall section although the thickness was not particularly large (~90 cm). The same results were obtained with sonic test. Later on the flat-jack tests were carried out at same wall to measure the value of the local vertical compressive stress and the stress-strain behaviour of the material. Finally coring of the wall at that position and laboratory tests on sampled material have been evaluated.

Positions 4a & b had almost the same state of compressive loading and morphology of the walls. Thus in the vicinity of Position 4b, an in-situ shear test were carried out at the wall at Position 4a. In-situ shear test was accompanied by sonic tests that were carried out before and after the shear test in order to quantify test results of sonic tests on intact and severely cracked masonry material.

6.1 NDT tests

6.1.1 Radar investigation

The first inspection was made by radar profiles (Figs 4 and 5) that clearly showed a position where the signal could not pass through the wall section although the thickness is not particularly large (about 90 cm). Profile and time acquisitions were carried out with 1 GHz and 500 MHz antennas. The time acquisitions were performed in positions 5 to 10 placing a metal shield on the backside of the wall during the second half of the experiment. Moving from left to right positions, the shield reflection is absent (positions 5, 6, 7), then it appears as a weak signal (position 8) and finally as a good signal (positions 9 and 10). To show this transition, the radar data collected at positions 8, 9 and 10 are plotted in Figure 4b. Where the radar signal could penetrate, an average velocity of 11 cm/ns was measured which is a rather common value for a stone masonry.
of a void or a deteriorated masonry that results in a poor elastic response for the sonic test and in a high absorption effect for the radar test. Besides these low values, the sonic velocities found by means of direct sonic tests were ranging from about 1000 to about 1400 m/s. Higher values were found locally, as shown in Figure 6.

Direct sonic tests and tomographies were carried out also in Position 4a, before and after the execution of the shear test. The main aim of the tests was to characterize the wall morphology and to calibrate the values of sonic velocities on the undamaged and damaged masonry wall. Two horizontal tomographies and one vertical tomography were carried out. Figure 7 shows a scheme of the tests performed.

The mean values of sonic velocity found in the upper and lower horizontal tomographies and in the vertical tomography were respectively equal to 1260 m/s, 1500 m/s and 1080 m/s. From the tomographic rendering, it was possible to notice the presence of a non-homogeneous section, but it was not possible to detect the presence of distinct masonry leaves. Figure 8 shows the results found on the upper horizontal section.

The mean decrease of sonic velocity found in the upper and lower horizontal tomographies, after carrying out the shear tests, were equal to 45% and 32%. The corresponding decrease of shear modulus, calculated for the upper and lower part of the tested specimen from the elastic effective stiffness and the secant stiffness in the inelastic range, was respectively equal to 48% and 26%. A comparison was made also between the decrease of velocity found by means of direct sonic
tests on the three grids of test positions for the tomography, and the crack pattern survey carried out during the shear test. The results of this comparison can be seen in Figure 9.

6.2 MDT tests

In order to evaluate the mechanic characteristic of the masonry and to calibrate the results of the NDT, slightly destructive tests were carried out.

6.2.1 Flat-Jack tests

The flat jack results show a good behaviour of the masonry (Fig. 10b) with an elastic modulus around 1490 N/mm² and with the level of local compressive stresses of $\sigma_0 = 0.143$ N/mm² (analytically calculated $\sigma_0$ was 0.154 N/mm²).

6.2.2 Coring

Coring at the same position (Position 4b) where the flat-jack, radar and sonic tests were executed revealed non-homogenous structure of testing masonry. It consisted of rubble stone masonry with large gaps between stones filled with gravels lied in poor lime mortar (Fig. 11).

Additional laboratory tests on samples from mortar specimens revealed the low value of soluble Silica which imply the lime as a binder. The structure of the stone material revealed from the cored sample showed that it consists of two types of stone. Petrographic tests on stone samples, revealed Lithothamnium limestones with following characteristics: density 20 kN/m³, compressive strength 125 MPa and modulus of elasticity 50.9 GPa and Calcernit stones (dark colour) with compressive strength up to 250 MPa.

6.2.3 Videoboroscopy

Videoboroscopy shows a non homogeneous but rather solid masonry (Fig. 12), which confirmed the good quality of the masonry.

6.3 DT tests

6.3.1 In-situ shear test

In-situ shear test was carried on the wall (Position 4a) previously agreed with representatives of Institute for
the Protection of Cultural Heritage of Slovenia. Execution of in-situ test was essential for the obtainment of mechanical properties needed for the assessment of the seismic resistance of the Western Part of the Castle. Removal and partial destruction of the tested wall is presented in Figures 13 and 14.

Lateral load was imposed in cyclic manner (one cycle per predefined displacement), with increasing the amplitude of the displacement up to the limit state of the specimen. The failure of the specimen was predefined as the attainment of the 80% of the maximum lateral resistance in softening region. The results of lateral tests are presented in Figure 15 and Table 1, where: \( H_u \) — analytically calculated idealized lateral resistance of the element; \( K_e \) — effective stiffness of masonry element; \( d_u \) — ductility of the masonry element; \( f_t \) — tensile (shear) strength of the masonry and \( G \) — shear modulus of the masonry calculated from the elastic effective stiffness.

### 7 SEISMIC ANALYSIS OF THE WESTERN PART

For the seismic analysis of the Western part following assumptions were made: rigid horizontal floor diaphragm action, predominant first vibration mode shape, contribution of an individual wall depends on the lateral displacement attributed to that wall and the shape of its resistance envelope and the walls of composite section were considered as separate along the vertical joint (or cracks observed with crack pattern survey) between their parts. Calculations were performed in two orthogonal directions (Fig. 3).

Main steps in non-linear analysis were: the calculation of the weight of building and stiffness of individual walls, determination of base shear and its distribution among the walls according to their stiffness. Seismic resistance was calculated on the basis of assumed ultimate resistance mechanism, which includes the redistribution of action effects to individual walls according to the attributed ductility capacity (storey resistance envelope). And finally following the energy and ductility based bi-linear idealization of the relationship between the resistance and relative storey
Figure 16. Relationship between the resistance and relative storey drift of the storey for X and Y direction respectively.

Figure 17. Elastic limit state – Y-direction.

drift of the storey under consideration, the comparison with design shear provisions were made.

Parameters for seismic analysis that were considered in this numerical investigation were: geometry of walls (derived through geometric and crack pattern survey), modulus of elasticity of masonry and compressive strength of masonry (double flat-jack test), shear modulus of the masonry and ductility (in-situ shear test) and vertical stresses in the elements (analytically and single flat-jack test).

Following the results of analysis presented in Figure 16, the ultimate design seismic resistance coefficient for X (SRCdu = 0.576) and Y (SRCdu = 0.646) directions respectively, have been compared with the design base shear coefficient BSCd. In the case of Pišece Castle, for the design ground acceleration of 0.2 g and importance factor of 1.4, the BSCd was calculated as 0.47.

Although the seismic analysis of Western Part of Pišece Castle has revealed that its seismic resistance is satisfying (which should not be surprising, considering that the ratio of areas of all walls towards the overall area of critical section was over 20%), the numerical analysis have shown also some weaknesses of the structure (Fig. 17), like are the short walls along the corridor, walls with chimney flues and already cracked round walls at the north part.

The numerical investigation was carried out under several assumptions. One of these was the rigid floor action, which for the Western Part of Pišece Castle still has to be achieved, since the floors in that part are wooden one and only a few anchors for tying the walls in the floor level were noticed during the investigations.

8 CONCLUSIONS

A great deal of research is still necessary for the interpretation of the NDT results and for their correlation with the masonry characteristics. However, from this case study it can be concluded that both radar and sonic tests have revealed a hidden chimney flues at Position 4b. Results of flat-jack tests at same position have been analytically verified with the differences between measured and calculated values of normal stresses less than 10%. Results of sonic tests carried out at Position 4a, before and after in-situ shear tests were significantly different.

Results from flat-jack tests and in-situ shear tests were than incorporated in non-linear seismic analysis of the Western part of Pišece Castle. Following results of geometry and crack pattern survey, main structural elements of the building have been defined. The results of seismic analysis revealed some weak points of the structure as well as possible origin of the large vertical cracks at the round tower of the north part of the building.

The on site investigation procedures should be calibrated and controlled in order to verify their effectiveness and particularly the possible application to each peculiar masonry problem. Since no test is usually self-sufficient to give the requested information, the complementarity of the different tests (in-situ shear, sonic and radar tests, flat-jack, etc.) has also to be studied for the definition of the necessary physical and mechanical parameters of masonries.

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