NDT, MDT AND DT SUPPORTED NUMERICAL MODELLING OF HISTORICAL STRUCTURES

V. Bosiljkov[1], V. Bokan-Bosiljkov[2] and R.Žarnić[3]

[1] Assistant Prof., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Chair for Research in Materials and Structures, Jamova cesta 2, 1000 Ljubljana, Slovenia. vbosiljk@fgg.uni-lj.si

[2] Assistant Prof., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Chair for Research in Materials and Structures, Jamova cesta 2, 1000 Ljubljana, Slovenia. vbokan@fgg.uni-lj.si

[3] Assosiate Prof., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Chair for Research in Materials and Structures, Jamova cesta 2, 1000 Ljubljana, Slovenia. rzarnic@fgg.uni-lj.si

ABSTRACT

The knowledge of the structural behaviour of existing masonry requires a multilevel approach, with proper application of different diagnostic and assessment methodologies. The paper presents successful application of the results of in-situ tests on modelling the behaviour of historical structures. In order to obtain reliable material parameters for the application of different numerical models, different test methodologies a combination of destructive tests (DT), minor destructive tests (MDT) and non-destructive tests (NDT) were performed. Following this, two different models were applied for the assessment of seismic resistance of Pišece Castle – structural element model (SEM) for non-linear seismic analysis and 3D finite element model (FEM) for linear analysis by means of modal response spectrum analysis. The results of numerical analysis provided valuable information of the load-bearing capacity of structures, their seismic resistance as well as possible causes for the observed crack patterns.

KEYWORDS: numerical modelling, historical structures, SEM, FEM, in-situ tests

INTRODUCTION

The analysis of historical masonry constructions is a demanding and complex task. Primarily, because only limited resources are usually allocated for the study of the mechanical behaviour of masonry. Furthermore even though, that sophisticated numerical model is available and some previous in-situ or laboratory tests were made, still there might appear obstacles in using existing tools and knowledge. Usually, silent aspects are missing data regarding geometry and morphology of built masonry; large variability of mechanical properties and their characterization; large variability of mechanical properties, due to workmanship and use of natural materials; significant changes in the core and constitution of structural elements, associated with long construction period as well as some retrofitting work; existing damage in the structure is unknown; regulations and codes may be non-applicable.

In order to overcome these obstacles, the first step should be to determine efficient methodologies for the diagnosis and the assessment of present state of the structure by means of NDT, MDT and DT techniques [1]. Hence, even though reliable parameters from on-site investigations have been obtained successfully, appropriate choice of method for analysis could be another obstacle in successful revitalization of historical structures.

In general, the seismic assessment of historical 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. For masonry wall structures with regular layouts where story mechanisms can be predicted (buildings, palaces etc.) SEM may be sufficient. However, for more sophisticated structures (towers, churches etc.), only FEM analysis can give insight into structural detailing and state of stresses through the thickness of masonry elements. Nevertheless, one may find that FEM approach has many limitations in comparison with simplified procedures that can represent the current behaviour of rather complex structures more realistic.

Depending on the structural characteristics of the building, according to Eurocode 8 [2] one of the following two types of linear-elastic analysis may be used: a) the "lateral force method of analysis" for buildings meeting the conditions for their regularity; b) the "modal response spectrum analysis", which is applicable to all types of buildings. As an alternative to a linear method, a non-linear method may also be used, such as: c) non-linear static (pushover) analysis; d) non-linear time history (dynamic) analysis. Which among these methods will be used in numerical analysis of historical structure may primarily depend from the type of the structure (castle, palace, church, tower, bridge etc.). The layout and structural system of historical masonry structures may be the crucial factors in this decision. It is likely to be, that with different methods also the results may differ to some extent. Some of the methods such are "lateral force method of analysis" or pushover analyses for some types of historical buildings are just not applicable. From these reasons, recently most of the authors who analyse historical structures use finite element models (FEM). Within the modelling historical structures 2D or 3D elements are generally used, with different types of material models and different approaches in modelling cracks within masonry.

STRUCTURAL ELEMENT MODELS

SEM approximates the actual structural geometry more accurately by describing individual structural elements such as piers and walls. In the case of single or multi-storey 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. 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 this 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 stepwise 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 relative displacement of the mass centre to the centre of the storey stiffness) the displacements of individual SE are modified. SEM assessment of historical masonry structures can be effective only if storey mechanisms can be predicted from the structural layout and configuration of the building. In order to apply the SEM method, crucial step is accurate characterisation of SE's and determination of their stiffness and resistance.

Besides knowing the geometry and boundary conditions of masonry structural element it is necessary to determine the mechanical properties of masonry, such as shear modulus (*G)* and elastic modulus (*E)* in order to calculate the design elastic stiffness. Then, load bearing capacity of structural elements can be calculated assuming two limit states according to Figure 1, knowing mechanical properties of masonry, such as compressive (f) and tensile strength (f_t). Following then Turnšek's theory [4] where the masonry, is considered as an elastic, homogeneous and isotropic structural material and considering the level of precompression and the geometry of the element, shear resistance may be calculated as:

$$H_s = \frac{f_t l t}{b}\sqrt{1+\frac{\sigma_0}{f_t}}\ ;\ \ b=\frac{h}{l}\ ;\ \ 1.1 \le b \le 1.5 \tag{1}$$

while rocking resistance of masonry structural element can be obtained from:

$$H_r = \psi \frac{l^2 t}{h}\frac{\sigma_0}{2}\left(1-\frac{\sigma_0}{f}\right) \tag{2}$$

where h – height of element, l – length of the element, t- thickness of the element, σ_0 – design compressive stresses, ψ – coefficient that reflect type of boundary conditions and is for both fixed ends equal to 2, otherwise it is equal to 1.

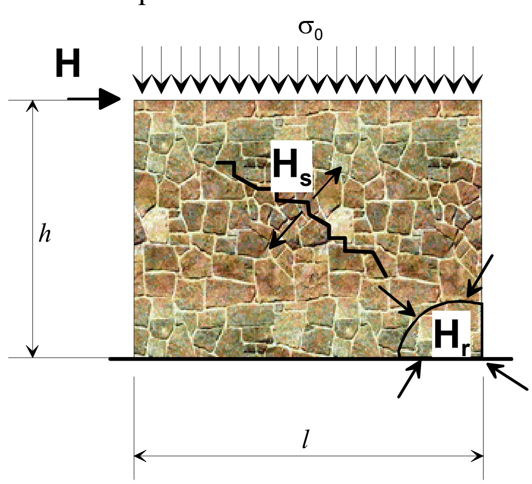

Figure 1: Limit states for in-plane resistance of masonry walls used in SEM analysis

Here it is necessary to stress that both equations 1&2 are not in accordance to Annex C of Eurocode 8 [5]. They differ from proposed equations in EC8 both in their analytical formulation, type of mechanical properties they are depend on, as well as confidence and material factors [3].

FINITE ELEMENT MODELS

In general the methods for the evaluation of the load bearing capacity of masonry structure and its elements are complementary and the selection of one of them depends on many factors, such as the required accuracy, measured parameters, the typology of the structural element, type of action, morphology of the masonry, etc. Masonry is a composite non-homogeneous structural material, whose mechanical properties depend on the properties of and the interaction between the composite components – units and mortar, their volume ratio and the properties of their bond. Furthermore, the properties and behaviour of masonry is strongly affected by the orientation of the main principle stresses towards the joints. It has to be said that FE procedures are not fully reliable when applied to historical masonry structures. One of the main difficulties to overcome is constitutive models for masonry and how to obtain material parameters for their implementation in modelling. Masonry is a typical non-linear material with reduced tensile strength for which cracking has be taken into account in order to describe the inelastic behaviour. While the choice of a very sophisticated constitutive model is important when the main objective of the analysis is to follow with the best possible accuracy the elastic, hardening as well as softening behaviour at all point of a structure, the simple elastic or plastic model may be accurate enough for the estimation of the global behaviour of the structure [6, 7].

RESEARCH PROGRAM

For the purpose of the study presented herein, two different types of structures were investigated: western wing of the Pišece Castle from Baroque period and its main defence tower from Romanesque period. Since both structures differ significantly both in the term of typology, state of structure and masonry texture, one of the main priorities was to organize qualitative on-site investigation of structures (Figure 2). The results of Non-Destructive (NDT), Minor Destructive (MDT) and Destructive testing (DT) provided reliable material parameters for the implementation of SEM and FEM analysis of both structures [1]. For the purpose of seismic analysis of western wing, non-linear static analysis was applied, while for the analysis of the main Tower, modal response spectrum analysis with linear elastic material was adopted.

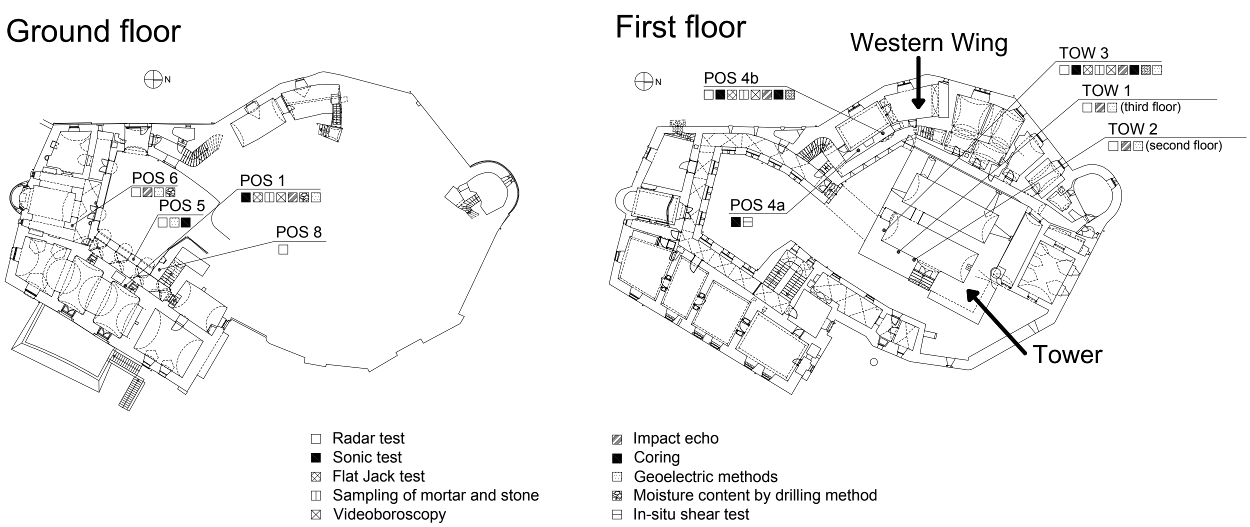

Figure 2: Organization of on-site investigation of the Pišece Castle

WESTERN WING OF THE PIŠECE CASTLE

From the structural point of view the Castle (Figure 3) represent classical example for castle in Slovenia. It consists of main defence tower, attached buildings around it, a chapel and defence wall around them. Structural elements that determine the load-bearing capacity of the castle are mainly solid walls, with the exception of the arched corridor connecting the main tower with the western part of the Pišece castle.

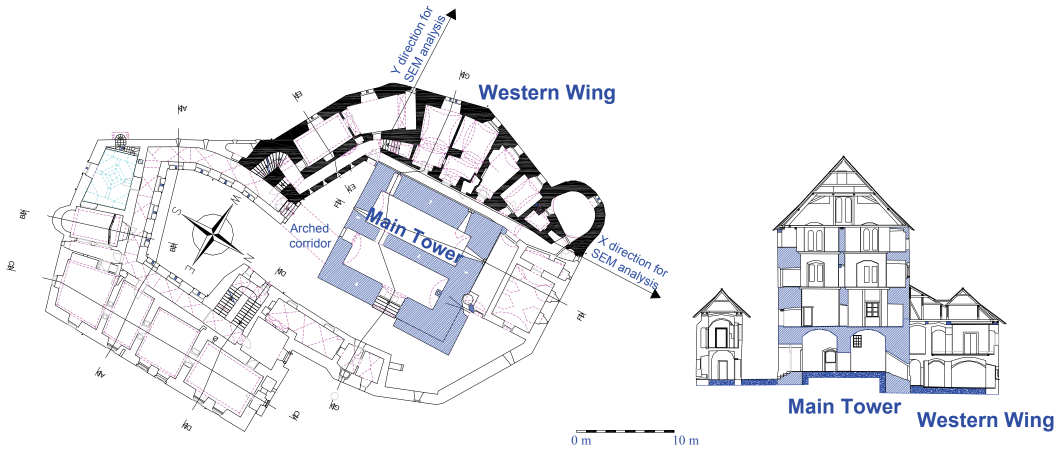

Figure 3: Structural layout and cross-section of western wing and the main tower of the Pišece Castle

For the western wing of the Pišece Castle, SEM model with applied push-over analysis was adopted for the verification of the seismic resistance. Parameters (Table 1) for the 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), tensile strength, shear modulus of the masonry and ductility (in-situ shear test), as well as vertical stresses in the elements (analytically and single flat-jack test) [see 1 and 8 for more details].

Table 1: Material parameters for SEM analysis

	Symbol	Value
Compressive strength (MPa)	f_c	1.00
Tensile strength (MPa)	f_t	0.14
Modulus of elasticity (MPa)	E	1490
Shear modulus (MPa)	G	430
Ductility	μ	1.0

For the seismic analysis of the western wing the following assumptions were made: rigid horizontal floor diaphragm action, predominant first vibration mode shape, contribution of an individual wall depend 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, determined by neighbouring massive defence

Tower, to which the western wing was leaning on. The main steps in the non-linear analysis were: calculation of the mass 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. And finally following the energy and ductility based bi-linear idealization of the relationship between the resistance and relative storey drift of the storey under consideration, the comparison with design shear provisions was made.

Following the results of analysis, the ultimate design seismic resistance coefficient for X (SRC_{du} = 0.576) and Y (SRC_{du} = 0.646) directions, respectively, was compared with the design base shear coefficient BSC_d according to EC8 [2]. In the case of the Pišece Castle, for the design ground acceleration of 0.2g and importance factor of 1.4, BSC_d was calculated as equal to 0.47. Although the seismic analysis of the western wing of the Pišece castle revealed that its seismic resistance according to EC8 requirements 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 showed also some weaknesses of the structure (Figure 4), such as short walls along the corridor, walls with chimney flues as well as already cracked round walls at the northern part of the wing.

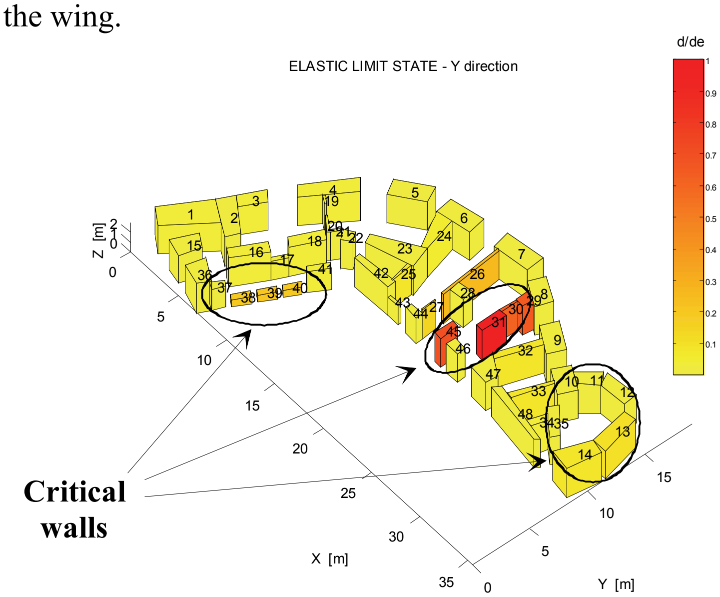

Figure 4: Elastic limit state – Y – direction

TOWER OF THE PIŠECE CASTLE

The main tower of the Pišece castle, which represents (beside the peripheral walls and the chapel) the oldest part of the castle, was built in the first half of the 13th century. It has rectangular shape with very (at some points almost 2.5 m) thick walls. It lay almost entirely on solid rock, except its south-east corner, where the soil was partly eroded. Originally the tower had ground floor and four floors above it, but in the 18th century the top floor was removed and the tower was partly attached with arched corridor to the western part of the castle, built later in Baroque period. The texture of the walls is of regular stone units distributed all through the thickness of the wall and thin bed joints. The connections between the walls are provided by masonry bond. The iron ties are located in the 3rd floor of the tower, but are in loose state, not providing satisfying additional connection of the main walls of the tower.

Crack pattern investigation (Figure 5) revealed that the tower is severely cracked all along the height of the east and partly south walls, with cracks on the east wall passing through the thickness of the walls. Also, the main inner partition wall is detached from the outer walls all along the height of the tower.

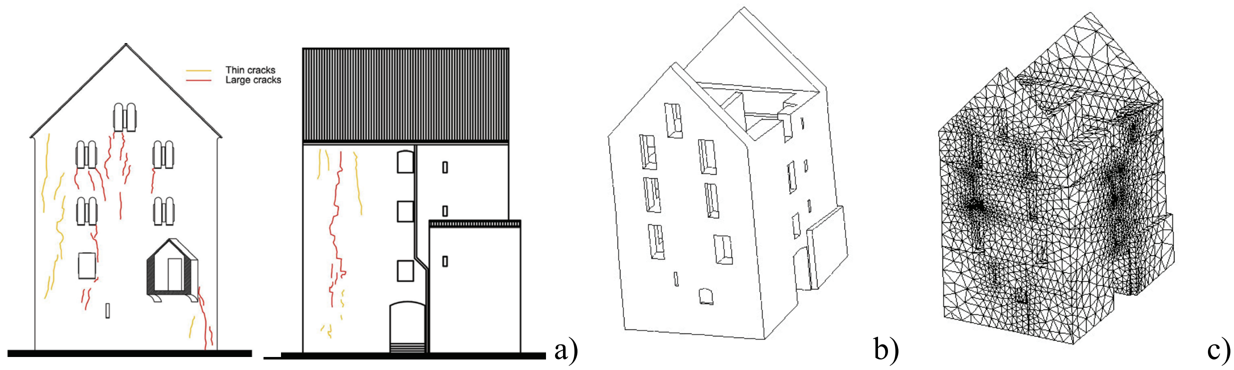

Figure 5: Crack pattern investigation of the Pišece Castle (courtesy of Politecnico di Milano) – a), geometry idealization – b) and mesh idealization – c)

For this stage of numerical analysis of the Pišece Castle tower, FEM macro-model with elastic material properties was applied. Application of SEM model was not feasible since storey mechanisms could not be applied for such massive structure (no concentration of masses at storey levels). The tower has also very wide walls and thus 3D FEM analysis of the state of stresses in critical sections was inevitable for successful analysis of the tower due to different load cases.

The mechanical characteristics of the masonry were formulated on the basis of the results from on-site testing campaign. The results from flat jack test as well as some general information gained through laboratory tests of cored samples were used in this numerical investigation. Specific weight was chosen as γ= 1800 kg/m^3 (although for regularly shaped masonry it may vary up to 2200 kg/m^3), modulus of elasticity E= 18 GPa; Poisson's ratio as ν = 0.03. Compressive strength was determined according to double flat-jack test as 1.2 MPa and tensile strength as 1/10 from compressive strength. Results of coring, videoboroscopy as well as radar and microseismic investigations [8] enabled modelling structural walls of the tower as homogenous isotropic material.

According to the crack pattern investigation there were several causes identified for the observed cracks, such as: a) due to the overweight of the tower, b) settlement of South-East corner and c) seismic motion (expected a_g = 0.20 [g] with return period of 475 yr.). Following this, at this stage of investigation a linear elastic analysis of the tower was carried out depending on different loading cases as presented in Table 2.

Two different models were prepared, named P and H. The first one (model P – see Figure 5) represents the model of the tower considering its current geometry. The second one (model H) was prepared by adding additional floor at the top of the tower, as it had been the situation till the 18th century. For dead load analysis only the dead load (no live loads) was considered (for roof structure of 0.9 kN/m^2 and for floor structures 0.372 kN/m^2). The settlement of the south-east corner of the tower was modelled as a linear elastic. The geometry of the tower was modelled considering all recesses and opening gaps. Constrains for the model were applied on the bottom of the model as well as at the connection between the tower and arched structure on the south side of the tower.

Table 2: Test matrix for different loading cases for Tower

	Present situation (XVIII – XXI)	Added top floor (XIII – XVIII)
Dead load	**PG**	**HG**
Settlement on South-East corner	**PSS**	**HSS**
Seismic analysis	**PS**	**HS**

Results on the PG and HG models revealed that the normal stresses at the most cracked part of the tower (south-east corner) do not exceed 30% of the compressive strength of the masonry. Thus the hypothesis that the overweight due to the added floor was the main cause for the cracking was neglected.

Another hypothesis for the extensive cracks at the south-east corner was due to the settlement of the tower. According to the analysis of shear stresses (see Figure 6), for both P and H model (load cases PSS and HSS), it seems that if the cracking was due to the settlement of the foundation on south-east corner, there should be extensive cracking along the main entrance to the tower at east side as well. Since, this was not the case, this hypothesis have been neglected as well.

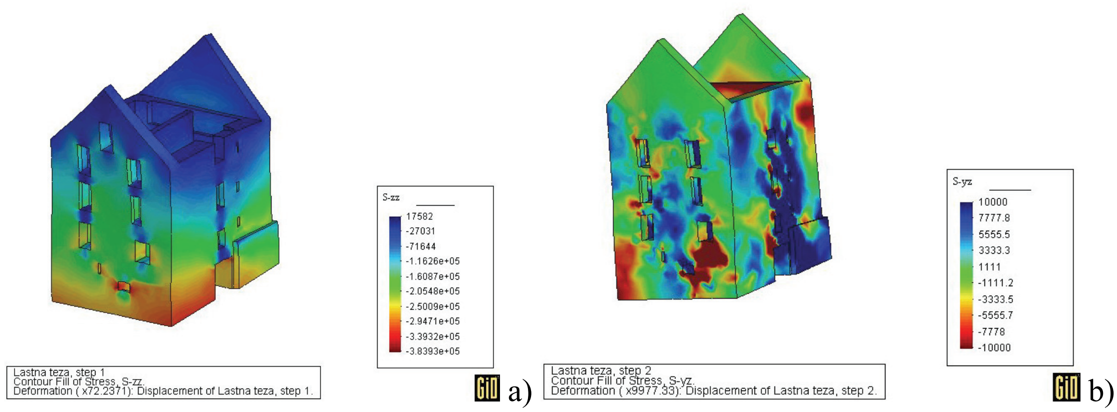

Figure 6: Load cases PG and HSS - Distribution of normal a) and shear b) stresses [N/m^2]

The seismic effects can conventionally be evaluated through static analysis of the structures subjected to a system of horizontal forces parallel to the directions assumed for the seismic motion and a system of vertical forces, distributed on the building proportional to the weights. For this seismic analysis, a seismic effect was evaluated through dynamic analysis of the building in the linear elastic field. This was done using the modal analysis method, adopting the seismic response spectrum. The method relies on the assumption that the dynamic response of a structure may be found by considering the independent response of each natural mode of vibration (Table 3 and Figure 7) and then combining the responses in a way that every possible result is given as an envelope of the maximum values of the nodal displacements, stresses, deformations and responses to the restraints, all calculated by combinations of the responses related to each type of vibration. For this paper the latest step was omitted and the analysis of each modal shape was considered separately.

Table 3: Results of modal analysis of Tower

Modal shape	Model PS Frequencies [Hz]	Model HS Frequencies [Hz]
1	11.42	14.38
2	13.26	16.26
3	17.61	20.21
4	21.38	26.31

Following the calculation of expected ground acceleration from EC 8 [2] seismic design spectrum that depends on the natural frequency for the mode of vibration under consideration and considering "A" type of soil, the seismic loads for the model PS was formulated. They correspond to the deformed shape depending on the mode of vibration under consideration with normalized displacement.

From the analysis of the distribution of the main tensile stresses for the model PS1, it can be concluded that the main cause for the cracking on the south and east side of the tower was due to the seismic actions (Figure 7-d). The seismic analysis of the 2nd and 3rd vibration mode revealed that these modes probably contributed more to emphasised cracking in the south part of the tower and damages on the gable wall.

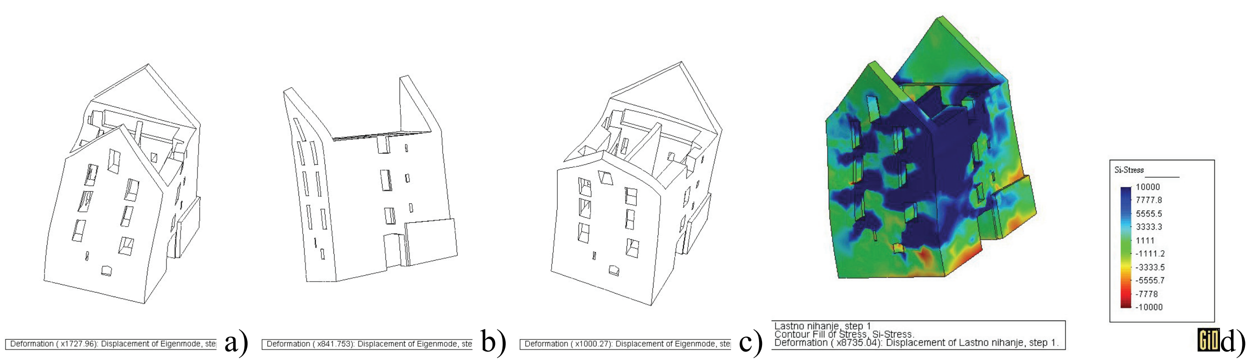

Figure 7: First three modal shape for model PS – a,b,c) and seismic analysis for the 1st modal shape – results for main tensile stresses –d)

CONCLUSIONS

Following the case study of the implementation of different NDT, MDT and DT techniques into the numerical investigation of the historical masonry structures several conclusions can be drawn:

- The results of NDT and MDT investigations can be used for the modelling of the elastic response of the historical masonry structures. However, for more sophisticated non-linear constitutive models, additional laboratory as well as on-site DT tests are required.
- Seismic resistance of western wing is according to the requirements of EC8. However, the numerical investigation using SEM method was carried out under several assumptions. One of these was rigid floor action, which for the western wing of the Pišece Castle still has to be achieved, since the floors in that part are wooden and only a few anchors for tying the walls in the floor level were noticed during the investigations.
- Numerical analysis of the tower revealed that the most critical load case for the safety of the structure is seismic load. Recorded crack pattern on the east and partly on the south walls of the tower were probably due to previous earthquakes. Concerning the crack beneath the corridor on the south wall (see Figure 5), it is more likely that it was the consequence of the later reconstruction works carried out in 18th century when the arched corridor was added.

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