BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iași
Tomul LXI (LXV), Fasc. 1, 2015
Secția
CONSTRUCȚII. ARHITECTURĂ

STRUCTURAL ASSESSMENT AND PROPOSED INTERVENTION WORKS ON A HISTORIC MASONRY TOWER

BY

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Received: February 10, 2015
Accepted for publication: February 21, 2015

Abstract. Studies concerned with evaluation and rehabilitation of historical masonry towers consider in situ investigations and structural analysis in order to better understand the seismic behaviour of these buildings, to define the causes for present damage and to asses the safety level for a large variety of actions. These pages follows, in the case of a historic masonry bell-tower structure, a description of the on site and laboratory investigations correlated with finite element models. Also, a proper retrofitting solution is presented. Advantages and disavantages of different modelling techniques, calibration of finite element models with ambient vibrations tests and the applicability of these methods in the field of old masonry structures assessment are presented.

Key words: masonry bell-tower; ambient vibration tests; modelling methods and calibration; intervention works.

1. Introduction

Preservation of the built heritage is considered an important aspect in modern societies as, in addition to their historical value, historical buildings

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significantly contribute to economy in a context where tourism has become a major industry. Their conservation, together with their monitoring and structural safety assessment, has become a main objective for European countries (Bartoli & Betti, 2013; Bowitz & Ibenholt, 2009).

Sf. Simion tower was made around the year 1551, as part of the Armenesti Church complex, having borowed the name and functioning as a belltower. The tower's height is 15.6 m (22.5 m at the top of the roof) and its dimensions in plain are 6.45 × 5.66 m. At the base, the tower is crossed (on the W-E direction) by an vaulted entrance, with the width of 2.70 m, therefore the tower's body is supported by two massive masonry piers. The cilindrical entrance vault is limited by two arched timpanyums, having the width of about 1.20 m. The foundations of the tower are composed by a solid block of 2.2 m width, made of stone masonry. The body of the tower is constituted of double walls filled with stone debris in a sand-lime-matrix.

Fig.1 – Sf. Simion's Tower.

The arhitectural and structural intervention works are due to the following main factors: the advanced degradation state, as a result of repetedly seismic actions and material ageing, the agressive vibrations on which the structure was subjected, vibrations caused by intense traffic, the necesity of structural and functional reabilitation for the use of local community (Fig. 1).

2. On Site Investigations

First, an accurate in situ survey of the tower configuration was carried out and included the survey of the crack patterns. The cracks were surveyed visually and photographically and reported on plans and sections. Furtheremore, the girders of the wooden scaffolding have been tested, specimens of stone and mortar extracted by core borings were analyzed in laboratory and ambient vibration tests were conducted in order to measure the dynamic response of the tower.

The wooden elements of the scaffolding have been in situ tested by using Pilodyn sclerometer in order to find the wood degradation depth. During testing (Fig. 2 *a*), a constant pin penetration, of about 14..15 mm, in the wood surface has resulted, which shows a superficial material damage depth of about 5 to 6mm. In two different points, a 16..17mm penetration depth was recorded, but due to local damage of the wood.

a *b*

Fig. 2 – On site investigation on St. Simion Tower: *a* – testing wooden elements with Pilodyn sclerometer; *b* – extracted mortar and stone specimen by core boring.

Core boring was performed at the right side facade wall, at a height of about 1m to ground level. The extracted core had 650 mm in length and 150 mm in diameter. The first 200 mm were stone representing the exterior part of the wall and the next 450 mm were stone debris in a sand-lime-matrix (Fig. 2 *b*).

After laboratory tests carried out on the extracted sample of stone and filling, the density, compressive strength and water absorption have resulted. A compressive of 16.1 N/mm^2 was obtained for stone specimen, 3 N/mm^2 for mortar and 10.3 N/mm^2 for filling material.

Ambient vibration tests were conducted at the bell and scaffolding level, placing the seismometer on two ortogonal directions: *x* direction (north to south) and *y* direction (east to west)(Fig. 3).

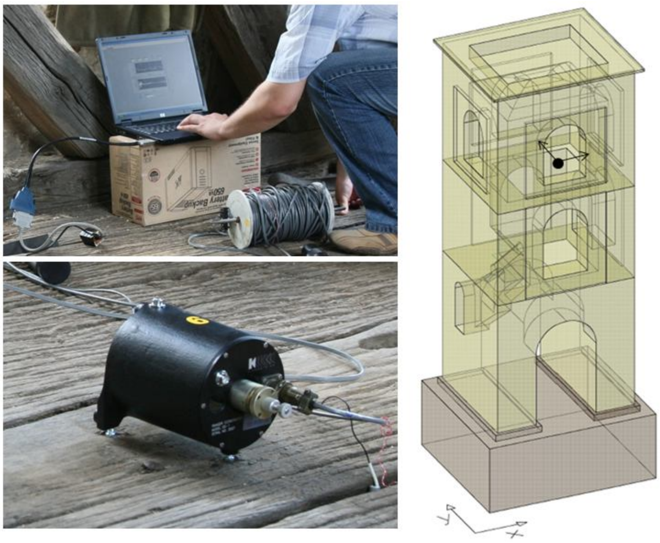
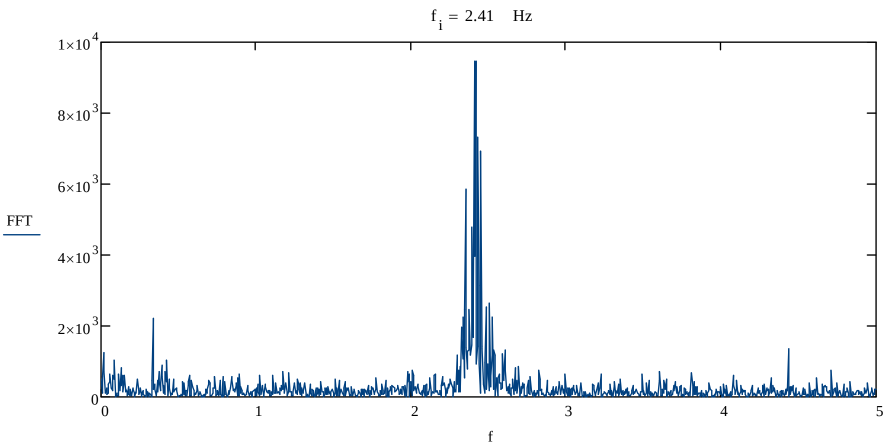


Fig. 3 – Ambient vibration test on St. Simion masonry tower.

The vibration measurements were performed with a seismic Kinematrics speed transducer, SS1 model, with a very high sensitivity (345 V/m/s) connected to a data acquisition system that converts the analog signal into a digital signal that is recorded on the hard disk of a computer. The signal was recorded for a period of 200 s, with a sampling period of 0.005 s (time between two readings).



f (Hz)

Fig.4 – Frequency response diagram obtained by FFT analysis for *y* direction.

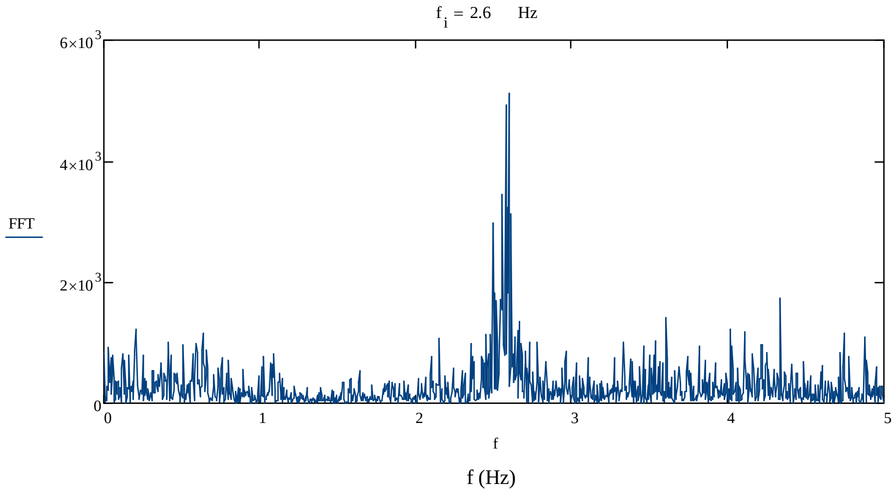


Fig. 5 – Frequency response diagram obtained by FFT analysis for *x* direction.

The measurement of the structure natural frequency was performed under the environment vibration, due to the nearby traffic. Frequency response diagrams obtained by FFT analysis for *x* and *y* directions are plotted in Fig. 4 and Fig. 5. On the specified diagrams frequency peaks were identified with the values 2.41Hz for *y* direction ans 2.60 Hz on *x* direction.

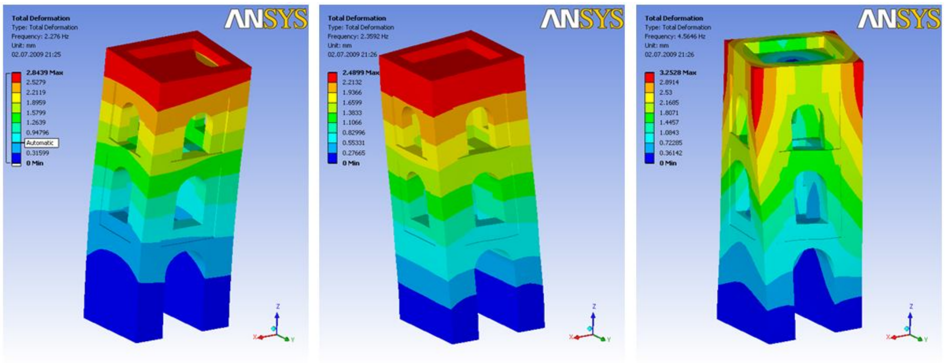
3. Finite Element Models

One of the basic step of creating an analytical model is creating a geometrical model. However, it is difficult to distinguish between the structural and decorative elements in case of historic masonry structures. As a general rule, the geometric idealization should be as simple as possible providing that the model is adequate for the problem being analyzed.

For example it would be unnecessary to use solid elements (Fig. 6) for the out of plane investigation of a masonry wall. Instead of solid elements, it would be enough to use shell elements for such a kind of investigation.

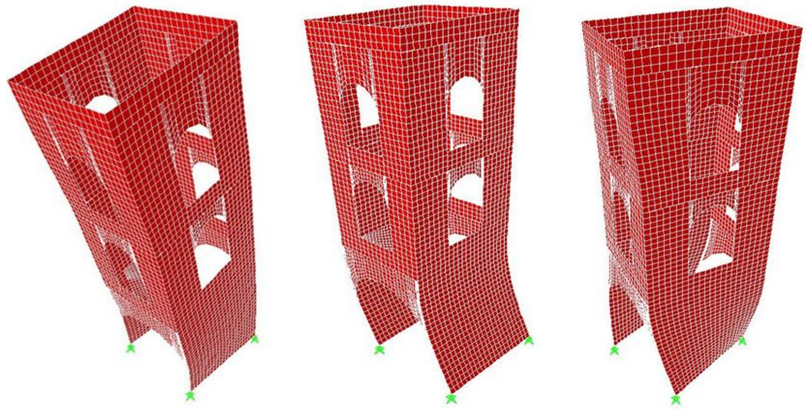
When the concern is the investigation of a thick wall for in-plane loading for example, using shell elements (Fig. 7) would be a bad choice since it would be very difficult to investigate the stresses through the thickness of the wall.

Modelling the tower masonry structure with solid elements had provided results regarding stress values and distribution inside the walls, providing significant informations on the structural response in gravitational and seismic loads.



1st Mode, f=2.36Hz **2nd Mode**, f=2.74Hz **3rd Mode**, f=4.56Hz

Fig. 6 – First 3 vibration modes of the structure modeled with solid elements.



1st Mode, f=2.31Hz **2nd Mode**, f=2.80Hz **3rd Mode**, f=4.51Hz

Fig. 7 – First 3 vibration modes of the structure modeled with shell elements.

In order to obtain the vibration modes and the distribution of the effort in the masonry walls, a three-dimensional model using shell elements has been developed. In the implementation of the computational model, an idealization of the geometry was necessary (modeling the arches and vaults with polygonal shaped surface elements, replacing the decorative elements and wooden scaffolding with loads corresponding to their weight). The vibration modes resulted from modal analysis are plotted in Fig. 7.

3.1. Calibration of the Models

Based on the modal analysis on two types of finite element models, using shell and solid elements, vibration modes with similar forms have resulted as following: the first mode is translation along the *y* axis (direction with a higher rigidity at the base level), the second mode is translation along *x* axis and the third mode is a torsional one.

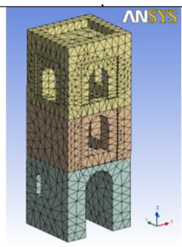
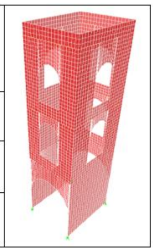
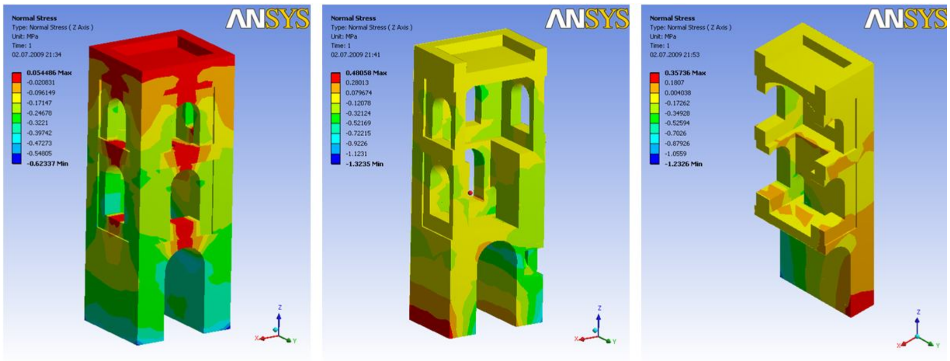
Vibr. mode	Experimental	Solid elements modelling			Shell elements modelling		
	f [Hz]	*f [Hz]*	*C*		*f [Hz]*	*C*	
Mode 1	2.41	2.36	**0.979**		2.31	**0.958**	
Mode 2	2.60	2.74	**1.053**		2.80	**1.076**	

Fig. 8 – The first two vibration modes frequencies and the coincidence of computational and experimental results after models calibration.

The calibration of the computational models was performed by affecting the masonry modulus of elasticity with a calibration coefficient computed as an average between the ratios of the frequencies resulted from computation and frequencies experimentally determined. A very good coincidence of computational and experimental frequencies have resulted (Fig. 8).

3.2. Displacements and Stress Analysis

By analyzing the stress maps (Fig. 9), a series of conclusions have resulted: in gravitational loads the entire cross section is compressed (the maximum compression stress is 0.6 N/mm^2), shear stresses have higher values



a *b* *c*

Fig. 9 – Compression stresses maps: *a* – gravitational loads; *b* – seismic action in *x* direction; *c* – seismic action in *y* direction.

in the walls weakened by the ladder acces niche, at horizontal seismic actions tensile stresses occur at the ends of the base cross sections (0.48 N/mm^2 on the north-south direction and 0.36 N/mm^2 on the east-west direction, spread over a small area at the ends of the section).

The maximum displacements resulted from finite element calculations are 15 mm on north-south direction and 14 mm on the east-west direction (along the legs of the tower), similar in size (approximately 1/1,000*height) for the first vibration mode.

3.3. Analytically Evaluation of the Safety Factors

A simplified approach, based on P100/3-2008 Romanian code, was used to manually compute the load bearing capacity. Three failure mechanisms of masonry walls are taken into account: rocking, sliding shear and diagonal shear cracking (Fig. 10). Rocking failure occurs when the moment at any of the end sections of the effective pier length attains the ultimate moment. According to P100/3-2008 ultimate moment equivalent shear force can be computed with the first equation from Table 1. The second criteria refers to sliding shear failure based on a Mohr Coulomb formulation and considers a simplified distribution of compression stress. The shear strength is given by the second equation from Table 1. The diagonal shear cracking strength for in plane actions is given by the third equation (Soveja & Gosav, 2014).

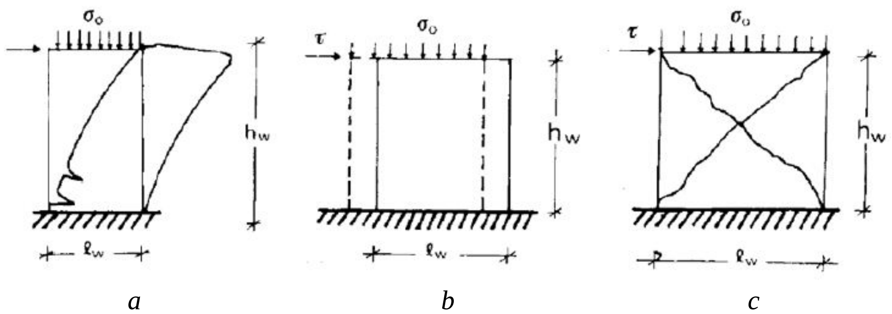


Fig. 10 – Failure mechanism of a masonry wall: *a* – rocking, *b* – sliding shear, *c* – diagonal shear cracking (Soveja & Gosav, 2014).

Table 1
Load Bearing Capacity for a Masonry Wall (P100/3-2008)

Failure mechanism	Equation
Rocking	$V_{f1}=\frac{N_d}{c_p\lambda_p}\nu_d(1\text{-}1{,}15\nu_d)$
Sliding shear	$V_{f21}=f_{vd}\times D^{'}\times t$
Diagonal shear cracking	$V_{f22}=\frac{t\times l_w\times f_{ctd}}{b}\times\sqrt{1+\frac{\sigma_0}{f_{ctd}}}$

Where: N_d is the walls axial force; λ_p – masonry wall form coefficient; ν_d – normalized vertical stress; f_{vd} – compressive design stress; D' – length of the compression zone; f_{td} – tensile design strength; b – coefficient with values between 1 and 1.5.

Following the computation scheme above analyzed, in Table 2 is presented the calculation of the safety factor for the two base walls of the analyzed masonry tower.

By comparing the bearing capacities with the shear forces resulted from the linear static analysis, using FEM, the safety factors for each masonry wall of the tower can be obtained.

The ratio of the structural capacity and seismic requirement, expressed in terms of strength, is the factor R_3 and represents the structural seismic safety level.

For the analyzed building, R_3 coefficient was calculated with a medium value of 0.142, which means that the first class of seismic risk could be assigned to the masonry tower.

Table 2
Safety Factor Computation (R_3)

Longitudinal direction									
Wall	t m	l_w m	N_d kN	V_d kN	V_{f1} kN	V_{f21} kN	V_{f22} kN	$V_{f\min}$ kN	R_{3i}
L1	1.77	5.50	4,701	2,670	582	1,126	1,952	582	**0.21**
L2	1.77	5.50	6,679	3,588	1,085	1,286	2,246	1,085	**0.30**
Transversal direction									
Wall	t m	l_w m	N_d kN	V_d kN	V_{f1} kN	V_{f21} kN	V_{f22} kN	$V_{f\min}$ kN	R_{3i}
T1	5.50	1.77	4,265	2,842	156	2,106	1,881	156	**0.05**
T2	5.50	1.77	2,491	2,066	57	3,162	1,559	57	**0.02**

4. Proposed Intervention Works

In the case of the analyzed masonry structure, the proposed intervention works refers to the execution of reinforced concrete belts and topping on the upper part of the vaults in order to improve the masonry walls confinement and the rigid diaphragm effect (Fig. 11). At the four corners of the towers, sets of post-tensioned steel ties in vertical bored holes should be executed.

The vertical ties have the role to induce permanent vertical compression stresses in the masonry walls, even in the case of seismic action and to improve the shear strength of the walls. The steel bars should be protected against corrosion resulted from the contact with the existing lime mortar.

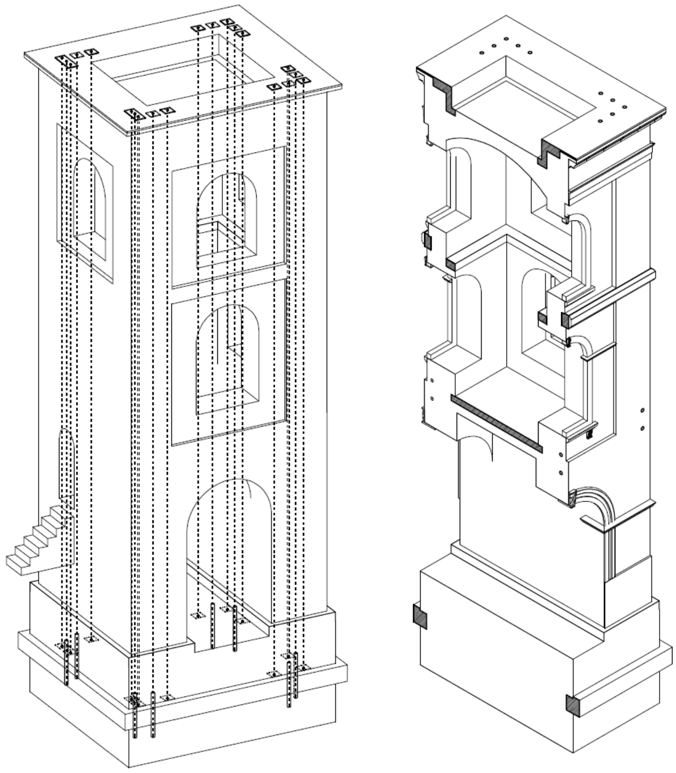


Fig. 11 – The sketch of the proposed intervention works on the masonry tower structure.

5. Conclusions

Investigation and intervention works on a historic masonry bell-tower are described in this paper. The following conclusions can be drawn from the study:

a) a superficial material damage depth of about 5 to 6 mm was shown by in situ tests of the wooden scaffolding using Pilodyn sclerometer;

b) the interior wall composition and materials density and compressive strengths where obtained by laboratory tests on samples of stone and mortar extracted with core boring;

c) after performing ambient vibration tests, frequency response diagrams were obtained by FFT analysis on which the frequencies of the first two vibration modes where identified on both directions;

d) two finite element models have been developed using only solid elements or shell elements; the calibration of the models is based on frequencies resulted from the ambient vibration tests; a very good coincidence of computational and experimental frequencies have resulted; vibration modes with very similar forms have resulted in the two corresponding modal analysis;

e) after stress and efforts distributions analysis, intervention works have been proposed corresponding to the damage state and structural features of the historic masonry structure;

Both assesment methodology and intervention works presented in this paper (based on in situ tests and finite element modelling) seems a promising approach to the evaluation and rehabilitation of such structures with a widely spreading throught the country.

REFERENCES

Bartoli G., Betti M., *In Situ and Dynamic Investigations on the Torre Grossa Masonry Tower*. Eng. Structures, **52**, 718-733 (2013).

Bowitz E., Ibenholt K., *Economic Impacts of Cultural Heritage – Research and Perspectives.* J. Cultural Heritage, **10**, *1*, 1-8 (2009).

Soveja L., Gosav I., *Analytical Evaluation of the Seismic Vulnerability for Masonry Churches Using the Finite Element Method.* Bul. Inst. Politehnic, Iaşi, **LX (LXIV)**, *3*, s. Constr. Archit., 55-66 (2014).

* * * *Cod de proiectare seismică.* Partea a III-a. *Prevederi pentru evaluarea seismică a clădirilor existente*. Indicativ P100/3-2008.

EVALUAREA STRUCTURALĂ ŞI LUCRĂRILE DE INTERVENȚII PROPUSE LA UN TURN VECHI DIN ZIDĂRIE

(Rezumat)

O mare parte din turnurile din zidărie monumente istorice se aflǎ în prezent într-o stare avansată de degradare structurală care poate conduce la neîndeplinirea cerințelor esențiale de calitate ale acestor tipuri de clădiri. Investigațiile în situ şi analiza structurală contribuie la evaluarea siguranței şi proiectarea lucrărilor de intervenții necesare, prin urmare sunt necesare rezultate precise, în scopul de a evita soluții de reabilitare nepotrivite. În această lucrare, se prezintă metode de investigare, analize numerice şi lucrări de consolidare într-un studiu de caz al unui turn vechi din zidărie monument istoric. Se prezintă avantajele şi dezavantajele modelării cu elemente de tip solid şi de tip shell, calibrarea modelelor de calcul pe baza măsurătorilor dinamice în situ şi o soluție de consolidare conform stării de degradare identificate. Metoda de evaluarea şi lucrările de intervenție propuse se dovedesc o abordare potrivită în cazul turnurilor vechi din zidărie de piatră şi emplecton reprezentand o tipologie constructivă des întalnită pe teritoriul țării.