SEISMIC STUDY OF HISTORICAL CASTLES AND THE EFFECT OF SOIL-STRUCTURE INTERACTION : CASE STUDY OF THE CASTLE OF BASHTOVA

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ABSTRACT

Historical buildings and monuments of built heritage in Albania suffered great damages during the earthquake of 26 November 2019. Most of them have a structural system of traditional load bearing stone masonry. Field observations and the overview of past interventions of several objects, suggested that the reported damages might have underlying causes, which triggered the main damages during the earthquake. This article focused on the study of damages of stiff masonry walls in castles and their relationship with the earthquake. We took as a case study the Castle of Bashtova, which is one of the few Albanian Castles located in flat terrain and on poor soil conditions. The purpose of this paper is the study of the link between the recorded damages through the history of the structure, the reported damages and the numerical analysis performed for the purpose of structural consolidation interventions. The analysis performed was the linear dynamic analysis with the response spectrum procedure for the calculation of stresses, displacements and their distributions, along with checks of the overall stability of the most critical sections with the Virtual Work Principle. The physical and mechanical properties of the materials were determined in laboratory and in-situ. The study showed that most of the damages reported after the earthquake were due to local effects of soil deposits or soil-structure interaction, local degradation of masonry and not sufficiently connected walls of different phases of construction.

KEYWORDS: Soil-structure interaction, linear dynamic analysis, response spectrum

INTRODUCTION

Bashtova Castle is located in a coastal lowland, very close to the delta of river of the Shkumbini River. The castle is very rare in its kind because it is a field castle, contrary to most of the other castles of Albania which are located on top of hills. Due to its location nearby the river and in a seismic area, during centuries, many damages have occurred. Except of partial repairs during 1980's the castle has never been restored. The earthquake of 26 November 2019 brought to the focus of engineers the need of assessment of structural

health, structural analysis and best restoration techniques for cultural heritage objects. The authors of the paper, which took part in the structural consolidation project of the castle, observed the main damages and related root causes of structural deterioration, including geological problems. Standard method of structural evaluation MEDEA was used for the preliminary evaluation of root causes of parabolic cracks related to differential settlement, vertical cracks related to different phases of construction or local compression, cracks between the openings of masonry due to seismic actions etc. However, to have a scientific and numerical evaluation of damages and to compare with in-situ observed damages, Finite Element Models FEM models were built. The effect of soil-structure interaction was observed, to have a better understanding of the effect of soil modelled as a linear spring in large FEM model. Furthermore, detailed analysis of equilibrium of walls were made to check their stability during earthquakes. It was observed that the damages comply with the weakest parts of the structural models, where the shear/tensile stresses exceed capacity. Since the castle is a 1st category monument of culture, the lowest impact methods of structural consolidation were proposed, starting from the survey of settlements, mortar repointing, Dutchman method of stitching masonry, and depending on long term observations even underpinning of existing foundations.

Construction Sector in New Normality

The entire earth is under the influence of the Novel Coronavirus or the Covid-19 pandemic since 2019. Progress towards the Sustainable Development Goals has stalled, and in some cases may have reversed. Up to 100 million people have slipped into extreme poverty – the first rise in global poverty in more than two decades (WHO, 2021). Construction industry is part of this crisis.
The new normality forces any kind of construction activity from the phases of design to implementation and service to take precautions versus the spread of the virus. Developing new mediums of working and application of these new principles will definitely reduce the and eventually stop the outbreak.

SITE CHARACTERIZATION, STRUCTURAL DESCRIPTION, MATERIALS AND SEISMIC INPUT

Site Characterization

Geological conditions and foundations.

According to the Geological Report and field observations, the castle rests on lands with very weak geological and engineering properties. The river deposits are composed of sands and silts, therefore the soil layers where the base of the castle foundations rest on have very poor physical-mechanical characteristics from the viewpoint of building construction. Consequently, any damage to the castle and its possible causes have been carefully assessed. A parabolic crack was observed in the north-east tower (the surface of the tower is cylindrical), which is related to the differential settlements, as well as a sloping crack in the wall perpendicular to the tower. This is an obvious sign of differential settlements (we have no historical information as to when this crack appeared). Another phenomenon is the deposition of river material after floods caused by the outflow of the river. According to foundation surveys conducted in the 1970s, the ground level in the 70's was about 170 cm, above the original building quota. In these conditions, although about 600 years have passed since the construction of the structure and the consolidation settlements would have been

finished, there is a need to monitor the possible settlements and cracks as well as to perform the analysis of the settlements, taking into account factors such as current geological and induced pressures, maximum and minimum groundwater level. No signs of liquefaction potential were observed. Since the terrain is flat, the possibility of potential landslides is inexistent.

Structural Description

The shallow rigid foundations of the building are built of irregular massive stones, bonded with lime mortar. The masonry built on the foundations is also built with stone blocks connected with lime mortar, having a thickness of 140 cm in the towers, while the walls between the towers having a thickness of 70 cm. Arches that serve as buttresses support the longitudinal walls; they are 70 cm wide and 140 cm thick. The current height of the towers is 8.3m, while that of the walls is 6.3m. The shape of the castle is quadrangular, but not a regular rectangle, however its western wall is totally detached from the rest of the castle due to the lack of two corner towers. This wall belongs to another phase of construction / reconstruction; the construction technique is distinct from the original. Even half of the north wall is built with the same later technique; there is even a quite distinct crack that corresponds to the interface of two different techniques.

Figure 35: Areal view (orientation) of the castle.

Materials Characterization and In-Situ Tests

The strength and strain parameters for each of the materials were taken in accordance with the textural description of the Atlas of Masonry and the Italian Technical Design Norms, updated in 2018. The authors of the paper will measure the in-situ stress, compressive strength and modulus of elasticity during the construction phase, because of the fact that the building is a monument of culture and performing tests require a special permit. Then, the FEM models will calibrate with the new physical and mechanical properties. However, for the purpose of the analysis of deterioration and structural proposals, the conservative data from the Atlas together the engineering judgment was used with a good accuracy.
The materials that compose the structure of the castle are stone and bricks connected with lime mortar Brick units are used only for the arches of the vaults. For both types of masonry, the physical-mechanical properties and stress-strain relationships are given below. Table 3. Strength parameters of the stone masonry.

Strain at the peak of resistance	ε_0	-0.0018	-
Limit deformation	ε_u	-0.003	-
Design compressive strength of masonry	f_d	-962.96	kN/m^2
Average value of the normal modulus of elasticity	E	1305000	kN/m^2
Tangent modulus of elasticity	E_{tan}	1003	MPa
Secant modulus of elasticity			
	$E_{(0,4fu-0)}$	949	MPa
	$E_{(0,2fu-0)}$	1013	MPa
	$E_{(0,4fu-0,2fu)}$	893	MPa

Table 4. Strength parameters of the stone masonry.

Stress-strain relations of irregular shape/carved stone masonry (foundations, walls, towers)

Table 2. Strain parameters of the stone masonry.

Specific unit weight of masonry	w	21 kN/m^3
Average compressive strength of masonry	f_m	2600.0 kN/m^2
Average shear strength of masonry	τ_0	56.0 kN/m^2
Average shear strength of masonry in the absence of normal stresses	f_{v0}	0.00 kN/m^2
Design compression resistance of masonry	f_d	962.96 kN/m^2
Design shear resistance of masonry	τ_{0d}	20.74 kN/m^2
Design shear resistance of masonry in the absence of normal stresses	$f_{v0,d}$	0.00 kN/m^2
The average value of the normal modulus of elasticity	E	1305 N/mm^2
The average value of the tangent modulus of elasticity	G	435 N/mm^2
Confidence factor	FC	1.35
Safety factor	γ_m	2

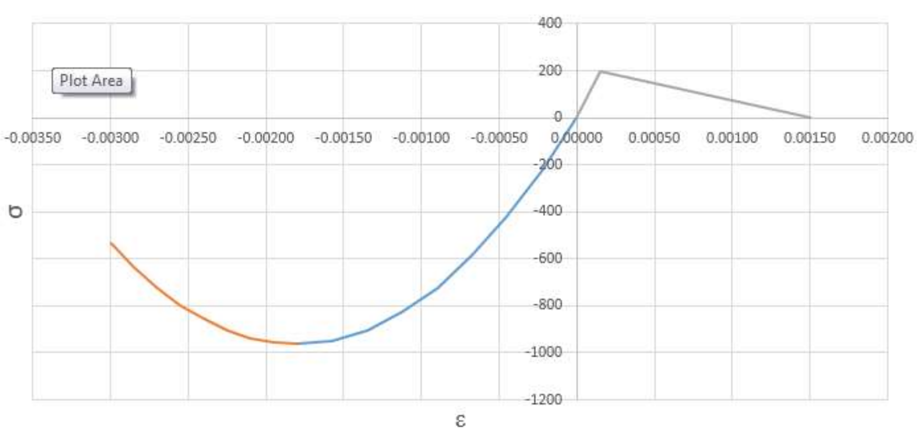

Figure 2: Stress-strain diagram of the stone masonry (stress unit kN/m^2).

Strength parameters		
Specific unit weight of masonry	w	18 kN/m^3
Average compressive strength of masonry	f_m	2400.0 kN/m^2
Average shear strength of masonry	τ_0	60.0 kN/m^2
Average shear strength of masonry in the absence of normal stresses	f_{v0}	0.00 kN/m^2
Design compression resistance of masonry	f_d	888.89 kN/m^2
Design shear resistance of masonry	τ_{0d}	22.22 kN/m^2
Design shear resistance of masonry in the absence of normal stresses	$f_{v0,d}$	0.00 kN/m^2
The average value of the normal modulus of elasticity	E	1125 N/mm^2
The average value of the tangent modulus of elasticity	G	375 N/mm^2
Confidence factor	FC	1.35
Safety factor	γ_m	2

Table 3. Strength parameters of the brick masonry.

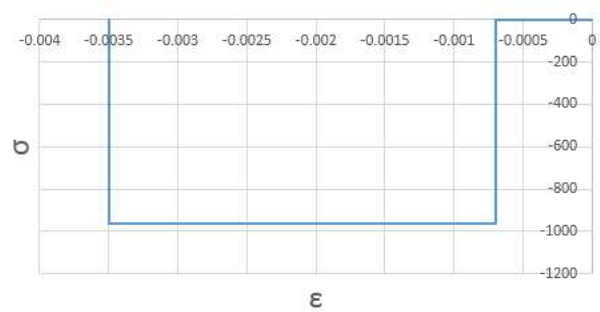

Figure 3: Design stress-strain diagram of the stone masonry (stress unit kN/m^2).

Stress-strain relations of brick masonry with lime mortar (arches of the vaults)

Table 4. Strain parameters of the brick masonry.

Strain at the peak of resistance		ε_0	-0.0018	-
Limit deformation		ε_u	-0.003	-
Design compressive strength of masonry		f_d	-962.96	kN/m^2
Average value of the normal modulus of elasticity		E	1305000	kN/m^2
Tangent modulus of elasticity		E_{tan}	926	MPa
Secant modulus of elasticity				
	$E(0,4f_u\text{-}0)$		876	MPa
	$E(0,2f_u\text{-}0)$		936	MPa
	$E(0,4f_u\text{-}0,2f^u)$		824	MPa

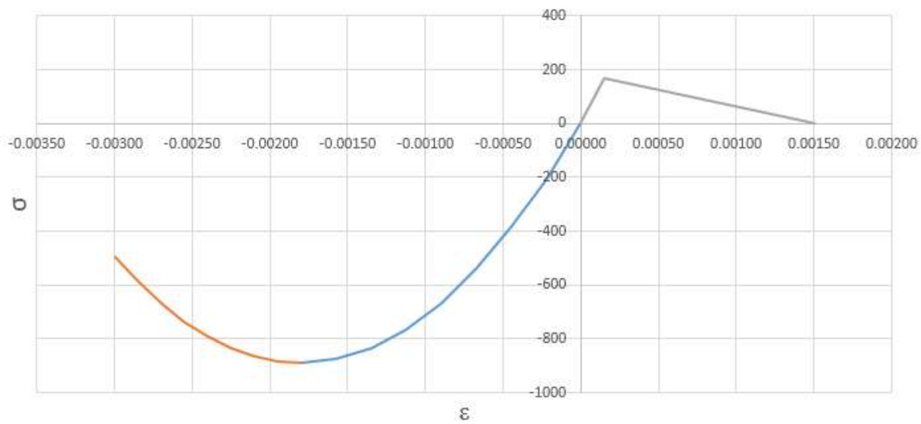

Figure 4: Stress-strain diagram of the brick masonry (stress unit kN/m^2).

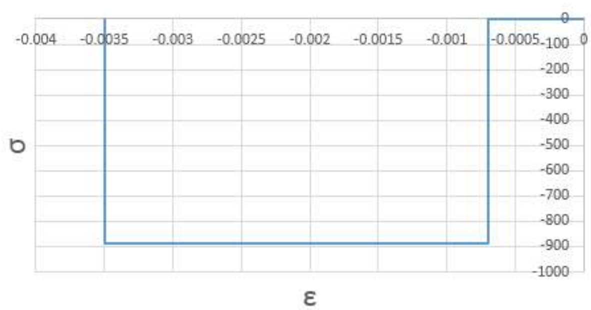

Figure 5: Design stress-strain diagram of the brick masonry (stress unit kN/m^2).

Seismic Input

The Seismic Report, for the Ultimate Limit State ULS considered a design earthquake with a repetition period of 475 years, with spectrum parameters PGA=0.278g, T_B=0.2s, T_C=0.6s, T_D=2.0s, S=1.15. Since the expected magnitude of the design earthquake is greater than 5.5, Type 1 of the reaction spectrum is used. According to the seismic report, the soil category is type C. Since the structure is a monumental work, then the Importance Factor of the structure is 1.2. Load combinations were obtained in accordance with Eurocode 1, which turned out to be:

Combination 1: **1.35 * Dead**
Combination 2: **1.35 * Dead + 1.5 * Live load**
Combination 3: **1.0 * Dead + 0.3 * Live load ± Spec.X**
Combination 4: **1.0 * Dead + 0.3 * Live load ± Spec.Y**
Combination 5: **1.0 * Dead ± Spec.X**
Combination 6: **1.0 * Dead ± Spec.Y**
Combination 7: **1.35 * Dead + 5 cm of settlement**

THREE-DIMENSIONAL SEISMIC ANALYSIS AND SOIL STRUCTURE INTERACTION

Structural Model

Mathematical modeling was built by means of the finite element method. This method consists in determining the internal force-deformation factors for rod-type elements, membranes, thin plates and thick plates, at the edges of elements of type "frame", "membrane", "plate", "shell". This method adapts to the two types of analysis allowed by Eurocode 8-3, static linear analysis and dynamic linear analysis-response spectrum method, for structures of First Class Knowledge Level, where the level of structure knowledge is limited to field tests, in the absence of original drawings and project specifications. Structural modeling and simulation were performed in the software SAP2000.

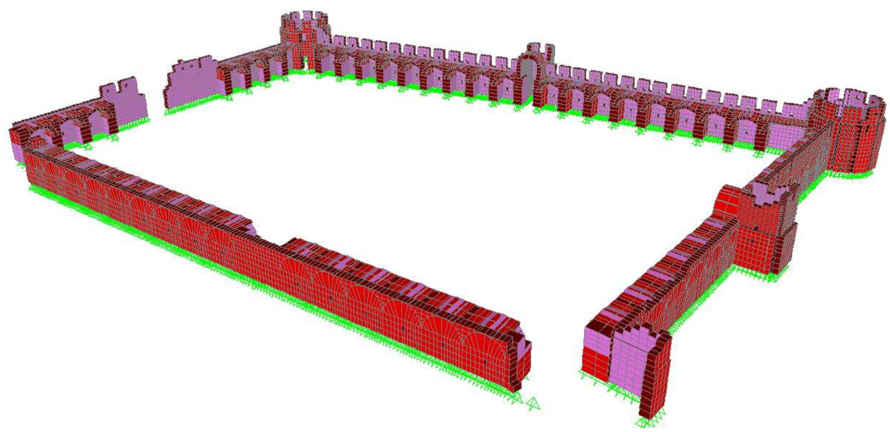

Figure 6: Structural model of the castle.

The foundations were modeled as partially fixed at the base of the building. The soil-structure interaction was taken into account through the modeling of the foundations as foundations on an elastic base, supported on elastic springs. The vertical masonry is modeled as a "thick shell" element, as it has a thickness of 0.7m. The modeling was performed taking into account that the walls are composed of two outer shells composed of carved stones and an inner wall filled with irregular stones and weak mechanical properties. Both materials and analysis were taken in the linear field. The analysis performed was the Linear Dynamic Analysis with the Response Spectrum Method. Three different structural models were built, since currently, due to the discontinuity of the masonry, the building consists of three different dynamic units: the main part (eastern, southern and half of the north wall), the western wall, and the northern wall. The Behavior Factor is accepted to be 1.5.

Modal Analysis Results

In order to have the highest possible mass participation ratio (80%), 25 mode shapes were taken into account during the modal analysis. The first three periods of the structure resulted as follows:

Table 5. Comparison of periods of vibration of three dynamic units.

Period No.	Castle – Main Part Period (s)	Castle – Northern Wall Period (s)	Castle – Western Wall Period (s)
1.	0.104	0.231	0.085
2.	0.102	0.087	0.087
3.	0.101	0.081	0.081

The analysis of the first periods of oscillations shows that the object has translational displacements and no torsion. According to Eurocode 8, an empirical formula for the approximate evaluation of the first period of the object is:
T=Ct x H3/4

where, H is the height of the building, while Ct for masonry buildings is 0.05.

Table 6. Comparison of periods of vibration of the tower.

Period	Eurocode 8 for the corner towers	Eurocode 8 for the walls
T_1 (s)	0.244	0.198

The above figures of the first oscillation period calculated with the empirical formula of Eurocode 8, shows that the modeling and simulation were accurate.

Settlement Analysis of The North-Eastern Tower

The differential settlement was applied in the part of the tower where the differential settlements were observed.

- **Horizontal tensile stresses**

The analysis of tensile stresses S_{11}, shows that the maximum values (2.30 daN/cm^2) are lower than the tensile bearing capacity of the masonry (2.8 daN/cm^2).

- **Shear stresses**

The analysis of shear stresses S_{12}, shows that the maximum values (1.42 daN/cm^2) are lower than the calculated shear capacity of masonry (5.6 daN/cm^2).

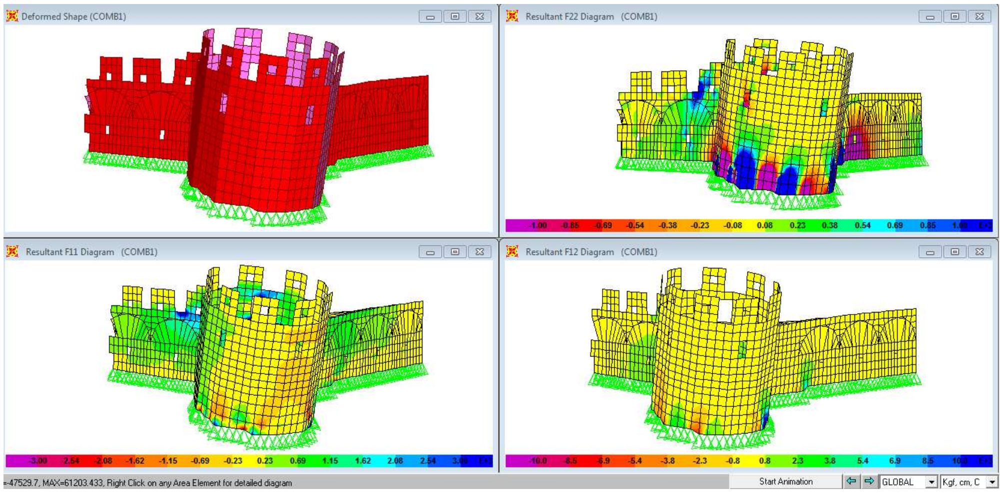

Figure 8: Tensile S_{11} and shear S_{12} stresses in the northeastern tower after inducing differential settlements equal to 5 cm.

Table 7. Stress check for the north-eastern tower (induced 5 cm settlement).

	Tensile stresses S_{11} (daN/cm^2)	Shear stresses S_{12} (daN/cm^2)
Northeastern tower	24.2 daN/cm^2	27.1 daN/cm^2
Strength	2.8 daN/cm^2	5.6 daN/cm^2
Strength check	Not satisfied	Not satisfied

The above values and the location of the concentration of tensile/shear stresses in the tower, clearly show that the settlement has caused the loss of shear / tensile strength of the masonry and explains the cause of the cracks that appear in this tower.

Figure 9: Existing crack of the north-eastern tower.

Stress Analysis

Analysis of the stresses in masonry walls after the consolidation of the foundations and masonry repair

- **Vertical compressive stresses**

Analysis of compressive stresses S22, shows that the maximum values (4.81 daN/cm2) are lower than the compressive bearing capacity of the masonry (260 daN/cm2).

- **Horizontal tensile stresses**

The analysis of tensile stresses S11, shows that the maximum values (2.30 daN/cm2) are lower than the tensile bearing capacity of the masonry (2.8 daN/cm2).

- **Shear stresses**

The analysis of shear stresses S12, shows that the maximum values (1.42 daN/cm2) are lower than the calculated shear capacity of masonry (5.6 daN/cm2).

Table 8. Comparison of stresses with the design strength.

	Compressive S_{22} (daN/cm^2) Envelope		Tensile S_{11} (daN/cm^2) Envelope		Shear S_{12} (daN/cm^2) Envelope	
	Occupancy	Earthquake	Occupancy	Earthquake	Occupancy	Earthquake
Northern wall	2.37	4.81	0.46	2.09	0.18	1.53
Western wall	2.41	1.02	0.27	0.43	0.24	0.77
Main part of the castle	2.42	1.61	1.21	2.30	0.57	1.42
Strength	260	260	2.8	2.8	5.6	5.6
Strength Check	OK	OK	OK	OK	OK	OK

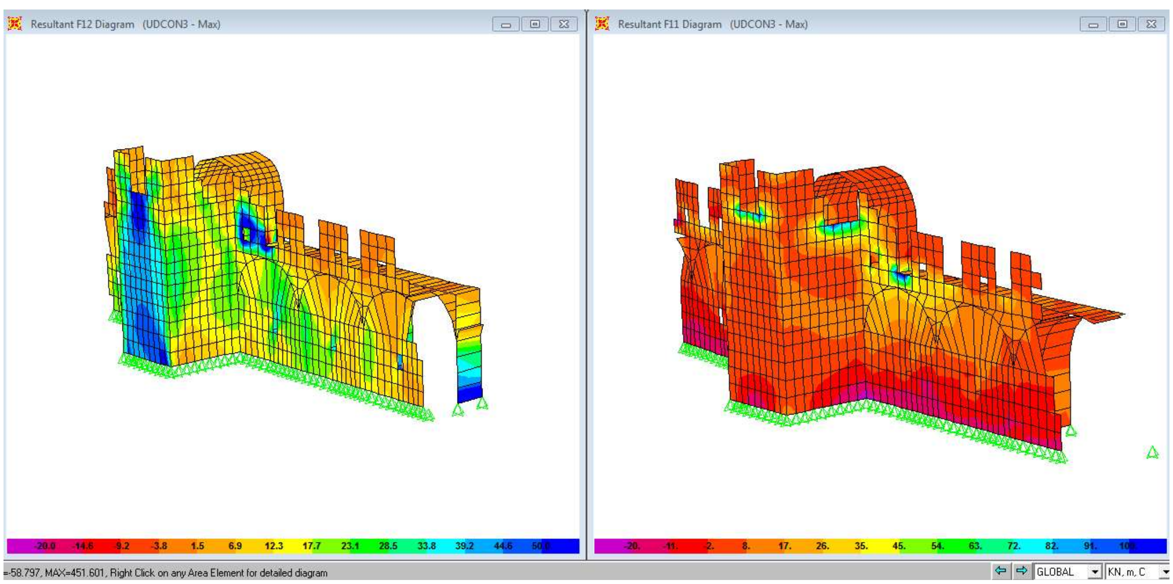

Figure 7: Tensile S_{11} and shear S_{12} stresses in the rectangular tower of the eastern façade.

Soil-Structure Interaction

The soil-structure interaction has a significant role on the seismic behavior of structure, mainly by modifying the fundamental period of vibration, and consequently the spectral accelerations and seismic forces. This interaction in the model has been modeled by linear springs following Winkler's approach of soil having elastic properties. The linear spring property, represented by the coefficient of soil subgrade modulus, was taken by the recommendation for etc. Determination of the exact values of the modulus of subgrade reactions with the Plate Load Test was not possible due to archaeological restrictions on excavations in the area of the castle.

Table 9. Typical values of soil modulus of subgrade reaction

	k: MN/m^3	
	Lower value	**Upper value**
Humus soil or peat	5	15
Recent embankment	10	20
Fine or slightly compacted sand	15	30
Well compacted sand	50	100
Very well compacted sand	100	150
Loam or clay (moist)	30	60
Loam or clay	80	100
Clay with sand	80	100
Crushed stone with sand	100	150
Course crashed stone	200	250
Well compacted crushed stone	200	300

For the purpose of this study, the median value of 45 MN/m3 was assumed to represent the elastic behavior of the loam/ clay (moist) soil. After running the analysis, results showed that the period of vibrations increased, while the stresses induced by the seismic forces decreased.

Period No.	Castle – Main Part Period (s)	Castle – Northern Wall Period (s)	Castle – Western Wall Period (s)
1.	0.550	0.663	0.736
2.	0.505	0.572	0.278
3.	0.491	0.400	0.182

Table 10. Comparison of periods of vibration of three dynamic units when soil-structure interaction has been taken into account.

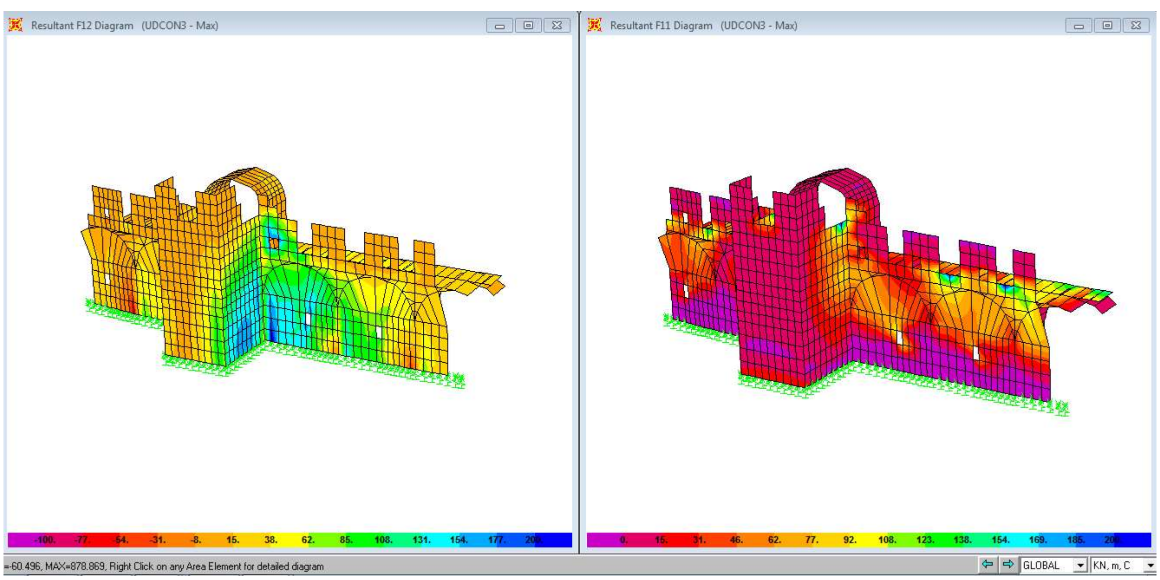

Figure 8: Tensile S_{11} and shear S_{12} stresses in the rectangular tower of the eastern façade when soil-structure interaction has been taken into account.

Analysis of the stresses in masonry walls when soil-structure interaction has been taken into account.

Vertical compressive stresses

Analysis of compressive stresses S22, shows that the maximum values (4.81 daN/cm2) are lower than the compressive bearing capacity of the masonry (260 daN/cm2).

- **Horizontal tensile stresses**

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- **Shear stresses**

The analysis of shear stresses S_{12}, shows that the maximum values (1.42 daN/cm^2) are lower than the calculated shear capacity of masonry (5.6 daN/cm^2).

Table 11. Comparison of stresses with the design strength.

	Compressive S_{22} (daN/cm^2) Envelope		Tensile S_{11} (daN/cm^2) Envelope		Shear S_{12} (daN/cm^2) Envelope	
	Occupancy	Earthquake	Occupancy	Earthquake	Occupancy	Earthquake
Northern wall	3.56	2.74	2.23	1.94	2.12	5.19
Western wall	3.37	1.43	1.23	0.46	1.02	2.64
Main part of the castle	3.90	2.25	5.31	2.47	2.63	5.13
Strength	260	260	2.8	2.8	5.6	5.6
Strength Check	OK	OK	OK	OK	OK	OK

SEISMIC CONSOLIDATION SOLUTIONS

The intervention will consist of two levels (local and global):
a. At the local level: repairing the masonry with the stitching technique (Dutchman method) or injecting cracks.
b. Locally: injection of traditional lime mortar into areas identified with cavities.
c. At the local level: improving the bearing capacity of the soil in the north-east tower area, in order to eliminate non-uniform settlements. The proposed method is to install piles in the area around the tower.
d. Globally: the interventions of points a, b, improve the inertial characteristics of the structure (as the discontinuity of the material is eliminated, the moment of inertia of the section increases and consequently the rigidity of the structure increases). In addition, eliminating the cause of the differential reductions will eliminate cracks and will have the same effect as that in the previous sentence. In addition, injection into the areas to be identified cavities increases the modulus of elasticity of the masonry.

CONCLUSIONS

The values obtained from the mathematical modeling of the maximum stresses and their location, matched the areas where the damages were observed. Tensile stresses had significant values in the areas where the masonry of the longitudinal walls intersects with the towers, as well as in the areas between the cracks of the windows in the masonry. In addition, the modeling of the differential settlement of the northeastern tower confirms the

presence of cracks in the structure, caused by exceeding the masonry tensile capacity. Soil-structure interaction seemed a significant role on the seismic behaviour of structure, mainly by modifying the fundamental period of vibration, and consequently the spectral accelerations and seismic forces. However, all the analysis and results were based on physical and mechanical properties recommended by the literature. During the construction phase, all the assumed properties, structural models and results will be calibrated by in-situ measurements.

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