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Modelling and Analysis of an Italian Medieval Castle Under Earthquake Loading: Diagnosis and Strengthening

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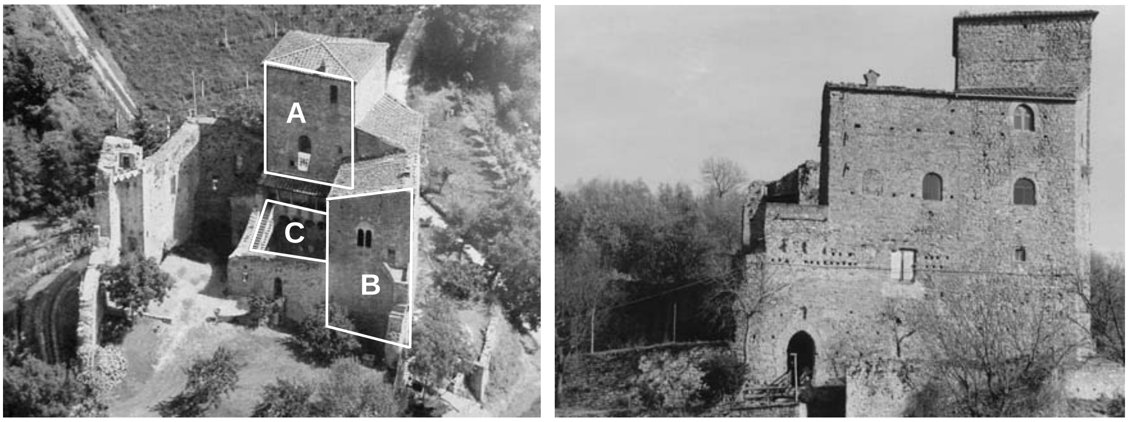
ABSTRACT: In this paper the Italian Medieval Castle of San Niccolò (Toscana, Italy) is analysed in order to assess its structural behaviour and its seismic vulnerability with respect to the actual state of conservation. A quasi-static approach for the evaluation of the seismic loads has been used (as indeed common in many analyses of the seismic behaviour of masonry structures). The comparison demand vs. capacity confirms the susceptibility of this type of buildings to extensive damage and possibly to collapse, as frequently observed. Herein, starting from a single case study, a contribution to the issue of modelling and analysis of monumental masonry buildings under seismic action is provided. The effects of the current techniques for repairing and strengthening are then investigated in order to evaluate the effectiveness of the usual retrofitting techniques.

1 INTRODUCTION

The analysis of ancient constructions poses important challenges because of the complexity of their geometry, the variability of the properties of traditional materials, the different building techniques, the absence of knowledge on the existing damage from the actions which affected the constructions throughout their life, and the lack of codes. As a matter of fact due to their intrinsic complexity, monumental buildings are by definition unique buildings, and they cannot be reduced to any standard structural scheme: this makes difficult to evaluate their seismic capacity. Nevertheless, significant advances occurred in the last decades concerning the development of adequate tools for the numerical analyses of historical structures.

If in principle, the prediction of the structural response of monumental buildings is not different from that of other structures (e.g. a bridge) it is an even more challenging task for several reasons. Each monumental building is by definition a unique building characterised by its own history, often resulting in a composite mixture of added or substituted structural elements, strongly interacting; the dynamic (and static, for that matter) behaviour of ancient buildings is normally too complicated to be interpreted by simple mechanical models. In particular train to extrapolate analytical procedures specifically developed for modern buildings is in most cases inadequate, since the static diagram is substantially different from the one of modern structures made of trusses and frames. Moreover it is quite difficult to perform reliable quantitative strength evaluations, due to the difficulty of gathering experimental data on the resistance of the structural elements and even on the mechanical properties of the materials on site. Another aspect that it's important to tale into account is that structural resistance of material decreases in time due to deterioration, and this degradation is frequently accelerated by neglect or carelessness. In brief, monumental historical buildings are by definition buildings that are difficult to reduce to any standard structural scheme because of the uncertainties that affect the structural behaviour and mechanical properties.

In this paper the Italian Medieval Castle of San Niccolò (see Fig. 1) is analysed in order to assess its structural behaviour and its seismic vulnerability. The Castle of San Niccolò is located over a mound near the right bank of the Solano river, in the neighbourhood of Strada in Casentino (Toscana, Italy). The castle consists of a great tower (A in Fig. 1a), a minor construction (B in Fig. 1a) and a courtyard (C in Fig. 1a). The building was affected by a diffused cracking pattern, so that from 1985 to 1990 some restoration works were performed: all main cracks were sealed, steel chains were put at three levels on the outer walls of the tower, the cross-vaults were consolidated through the creation of RC (reinforced concrete) beams over the diagonal ribs and RC slabs, a RC arch was built to support the external masonry stairs and four micro-piles were inserted under the masonry foundation of a column.



(a) (b)

Figure 1 : Castle of S. Niccolò : (a) Aerial view; (b) Frontal view

The structural evaluation of the castle under vertical and seismic loads has been performed in order to recognize the main cause of the cracking pattern, which has been identified in the differential settlements of the foundations. To this aim 3D non-linear analyses that take into account the non-linear behaviour of masonry have been performed: constitutive assumptions, characterized by elasticity, damage and friction, are done. The behaviour of the masonry is replicated by use of a solid element that can have its stiffness modified by the development of crack and crushing. The standard FEM modelling strategy based on the concepts of homogenizated material and smeared cracking constitutive law is used. An evaluation of the capacity of the castle to withstand lateral loads together with the expected demands from seismic actions is also given. The effects of the current techniques for repairing and strengthening are then investigated in order to evaluate the effectiveness of the usual retrofitting techniques.

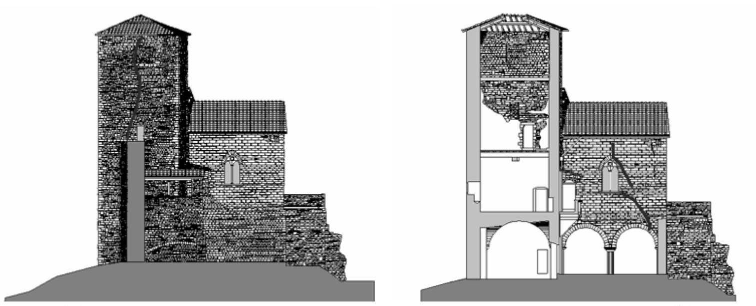
2 ACTUAL STATE OF CONSERVATION

The knowledge of the actual cracking path, and then the state of conservation of the masonry building, is a crucial task for the assessment of the vulnerability under earthquake loads. The Castle of S. Niccolò shows a variegate cracking pattern. The main one is interesting the great tower (see Fig. 2), other cracks are observable on the masonry wall and on the small building (minor construction that appears on the courtyard, see Fig. 2b and Fig. 3a). An analysis of the different cracks on the Castle has permitted a preliminary explanation of this damage. It is possible to attribute the crack pattern to a rotation of the great tower toward the courtyard (see Fig. 3b). This hypothesis is confirmed by two main aspects: the first one concerns the ground properties, the tower is founded on a rock area, on the contrary the other parts are founded on a alluvial deposit; the second one is connected to the presence under the courtyard of a tank, that reduce the stiffness of the area in front of the tower.

3 STRUCTURAL ANALYSIS

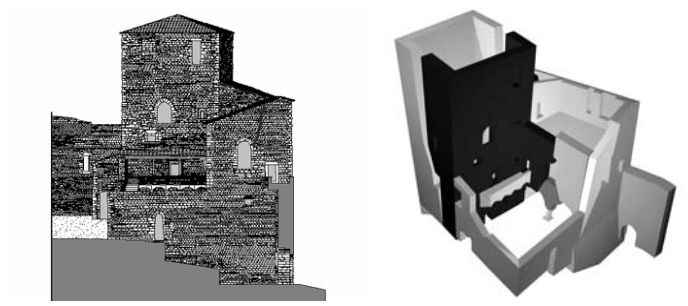
Analysis of the seismic behaviour of historic masonry buildings, and in particular of Castles, is a difficult task due to: the numerical modelling of the nonlinear behaviour of the masonry, with almost no tensile strength, the incomplete experimental characterisation of the mechanical properties of the masonry, the complexity of the geometry.

Refined mechanical models, which accurately predict the behaviour of masonry material and elements, have been proposed in the inherent literature Betti and Vignoli (2005), Del Piero (198), Grimaldi et al. (1992). Such models adopt different strategies to take into account the highly nonlinear behaviour of the material both in tension (low tensile capacity and consequent cracking phenomena) and in compression, and some of them are also able to provide the structure response to large cyclic deformations, which occur under seismic actions. Unfortunately, they are hardly applicable to the 3D analysis of complex structural systems, due to the great number of parameters involved in the definition and updating of the mechanical model and the large number of degrees of freedom required to mesh the structure, leading to untreatable problems.



(a) (b)

Figure 2 : (a) Cracking on Great Tower (Wall T4) : (b) Cracking on WallA3



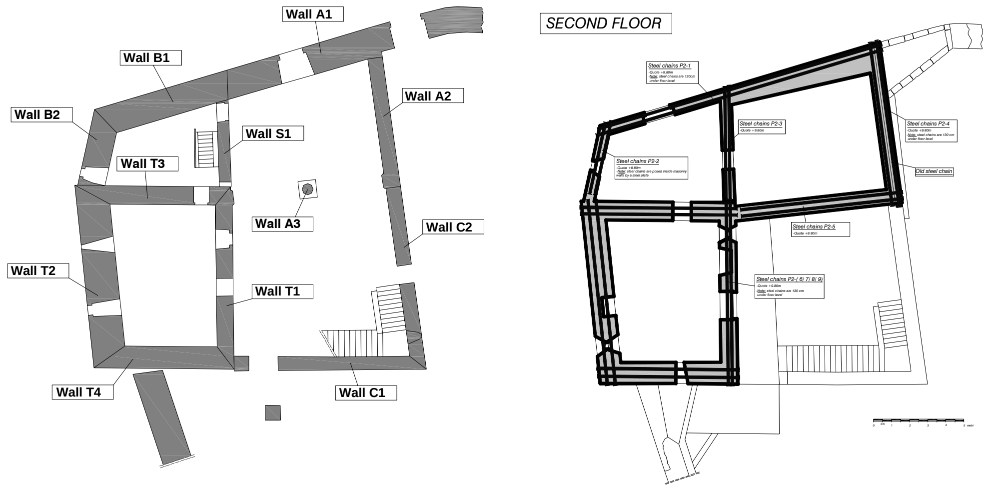
(a) (b)

Figure 3 : (a) Cracking on WallA2; (b) Reconstruction of general displacements

The analysis method proposed and adopted in this paper for overcoming the above difficulties is based on a three-step procedure: (a) firstly the overall structure is analysed in the nonlinear range through a complete and refined 3D model; some different hypothesis are made concerning the constraints on the ground surface in order to identify the numerical model and to understand the causes of the cracking pattern; (b) secondly, using the identified model, the structure is analysed with the aim of characterising the static and dynamic behaviour, defining the internal force distribution and identifying the weak points of potential failure in the building; the effect of the restoration works done in 1990 are then investigated in order to understand the actual state of conservation of the Castle; (c) thirdly the non linear model previously identified, including the modeling of the restorations works, is used for an assessment of the seismic behaviour of the whole building.

3.1 Non linear static analysis

Static and dynamic analyses have been carried out on the 3D model of the Castle structural complex using the F.E. computer code ANSYS. The masonry walls have been modelled by means of *Solid65* elements, while *Shell143* elements have been used to model the main vault at the first floor of the great tower. The 3D model consists of 20158 joints, 13515 3D solid45 elements. Material properties (Young modulus E, Poisson coefficient ν, own weight W) of masonry walls are differentiate taking into account each different area present on the building (see Fig. 4). For the masonry elements the hypothesis of non-linear elastic behaviour has been adopted. Masonry non linear behaviour is defined by selecting the yield Drucker-Prager criterion with associated flow rule. It is assumed that the yield surface does not change with progressive yielding, hence there is no hardening rule and the material is elastic-perfectly plastic. The failure surface assumed is the Willam and Warnke surface. Table 1 reports the selected values for the model parameters with respect a specified wall.



(a) (b)

Figure 4 : (a) Elementary walls; (b) Steel chains on second floor

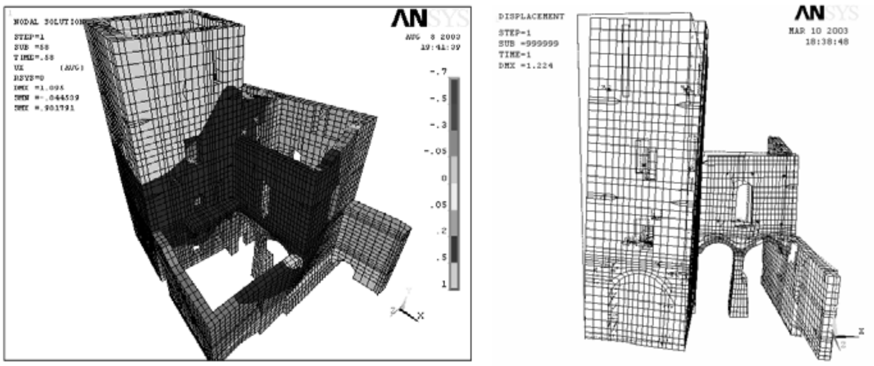
The structural elements have been analysed under constant vertical loads, deriving from the own weight and from the roof and floor loads. The main information which can be derived from this preliminary static analysis is the interaction between the Castle and the ground. A static analysis where the model is assumed to be connected with the ground with rigid constraints is done. Results of this analysis are not able to reproduce the cracking pattern on the masonry wall then, in order to attest the mechanism previously identified, a new model where link (combin14) are assumed on the ground has been carried out. Results, with respect to the walls shown in Fig. 4a, are reported in Figs. 5-7.

Table 1 : Yield criterion and failure surface setting (Wall A3)

Yield Drucker-Prager criterion		Willam and Warnke surface	
c (cohesion)	0.9 kg/cm^2	f_c (uniaxial compressive strength)	80 kg/cm^2
η (dilatancy)	15°	f_t (uniaxial tensile strength)	1.5 kg/cm^2
φ (internal friction angle)	38°	β_c (shear transfer coeff. close crack)	0.75
		β_t (shear transfer coeff. open crack)	0.15

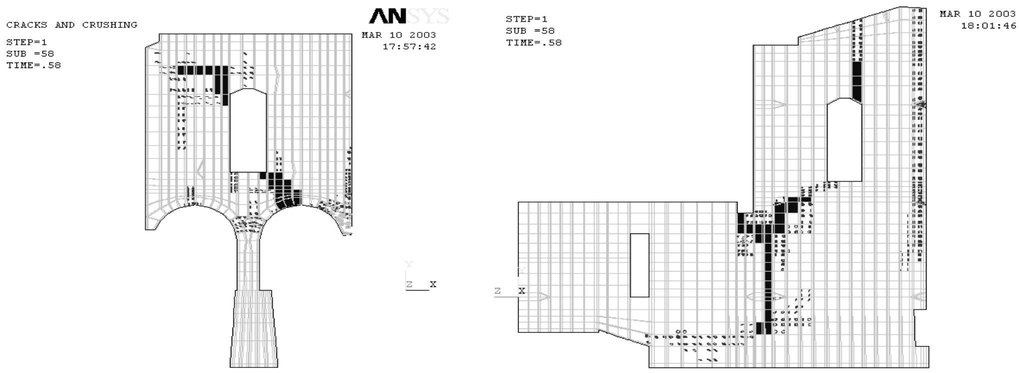
In Fig. 5 the results of the static analyses on the 3D model, under constant vertical loads deriving from the own weight and from the roof and floor loads, are reported in terms of displacements. As a matter of fact it is possible to read a general movement where wall T1 (see Fig. 7c) moves toward the courtyard. This movement originates a cracking path on the walls T4 (see Fig. 7b) and T3 (where masonry is not able to transfer tensile stresses) and it pushes against the walls A3 (see Fig. 6a) and C2/A2 (see Fig. 6b). By the non-linear analysis, with an iterative procedure, it has been identified the cracking path on the building; it is connected to two main components. The first one is due to the differences in ground properties, while the second one is related to the presence of a tank for rainwater under the courtyard.

The identified numerical model has been, successively, modified in order to take into account the restoration works done from 1985 to 1990. These works consisted in main cracks sealing and putting steel chains (see Fid. 4b) at three levels on the outer walls of the tower. Cross-vaults were consolidated through the creation of RC beams over the diagonal ribs and RC slabs. A RC arch was built to support the external masonry stairs and four micro-piles were inserted under the masonry foundation of a column (Wall A3). *Shell143* are used for the modelling of RC consolidations, and *link10* are used for modeling of steel chains. *Mass21* elements has been used for take into account the new loads deriving for the restorations of the floor. The 3D model consists of 21396 joints, 13709 3D *solid45* elements, 507 *shell143* elements, 403 *lonk10*, 577 *combine14* and 190 *mass21* elements.



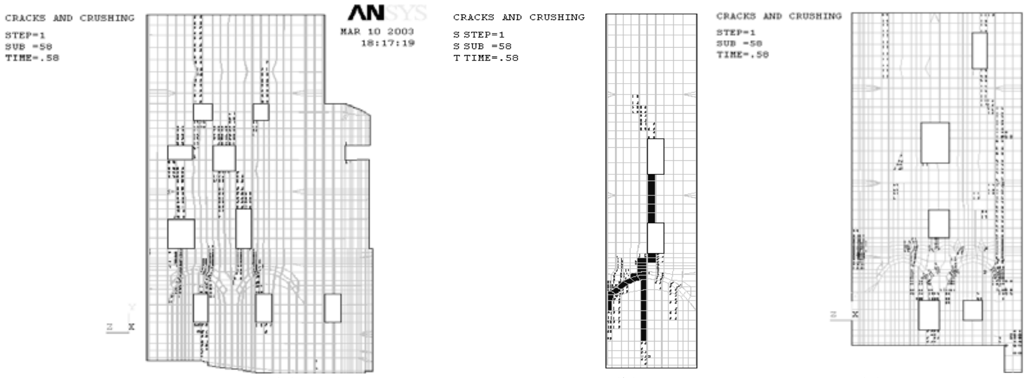
(a) (b)

Figure 5 : (a) Horizontal displacements; (b) Deformed shape



(a) (b)

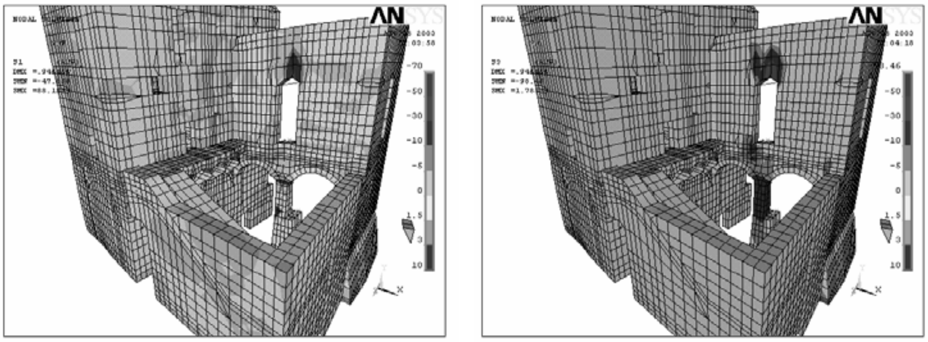
Figure 6 : (a) Cracking path on Wall A3; (b) Cracking path on Wall A2/C2



(a) (b) (c)

Figure 7 : (a) Cracking path on Wall T2; (b) Cracking path on Wall T4; (c) Cracking path on Wall T1

The maximum value of the tension is reached by the columns that sustains the WallA3 (see Fig. 8a), denoting a critical point on the construction; over the entire building the maximum value of the compression equals the masonry compressive strength. It is possible to observe, with respect to the displacements, a general benefit due to the analyzed restoration. These restoration works are not able to avoid the movement of Wall T1 against the courtyard but there is a general decreasing of both the displacements and cracking on the building. The results shows that a general benefit is produced by the restoration done in 1990; particularly advantage is done by the presence of the steel chains on the Tower even if these are not able to reduce the general movement toward the courtyard. Nevertheless the building still needs some restoration works that allow to reduce the underlined structural defects. A global micropiles restoration is needed in order to eliminate the differences on ground properties. This model in next paragraph will be used in order to assess the seismic capacity of the building.

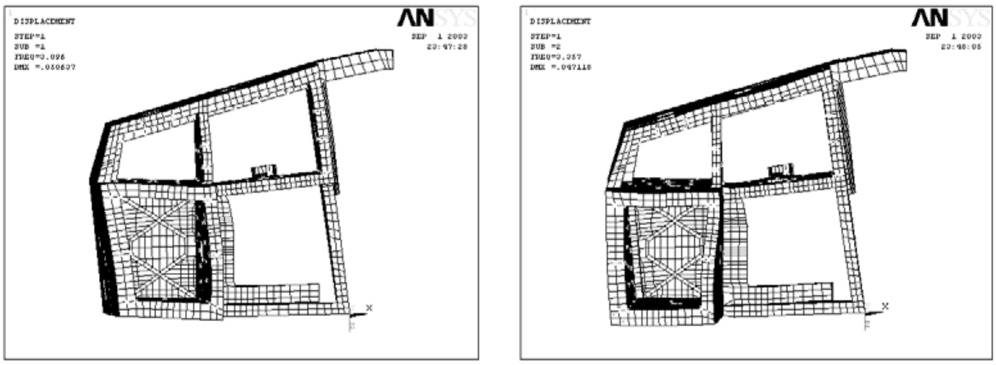


(a) (b)

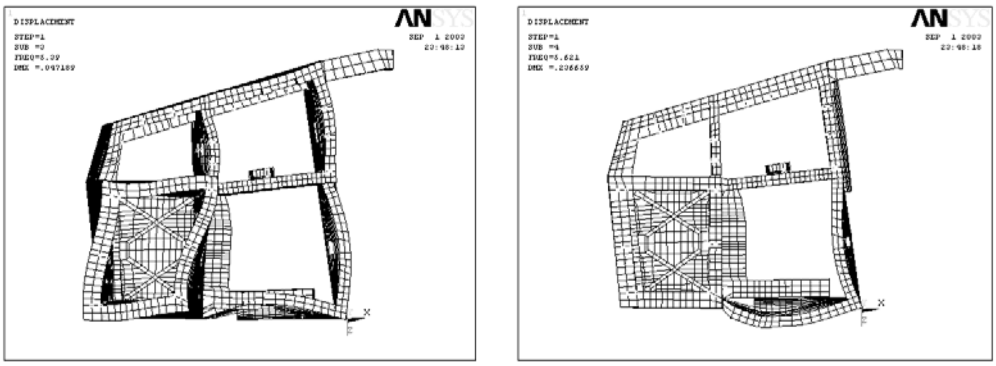
Figure 8 : Principal stresses [kg/cm^2]

3.2 Seismic analysis

In order to simulate the behaviour of the Castle under seismic loads the building was subjected to an equivalent static analysis through the application of horizontal forces perpendicular to one another. These forces, not acting simultaneously, are evaluated according to EC8.



(a) first mode T_1= 0.323 sec. (b) second mode T_2= 0.298 sec.



(c) third mode T_3= 0.186 sec. (d) fourth mode T_4= 0.178 sec.

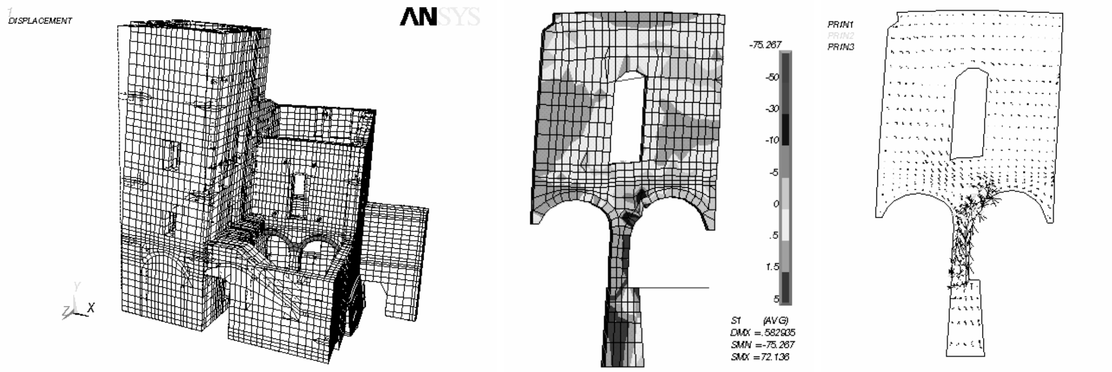
Figure 9 : Periods and mode shapes of the 3D identified F.E. model

A preliminary modal analysis where the effective cracking actual path is taken into account is done. Results concerning this preliminary step are reported in Fig. 9. Modal analysis is also used in order to evaluate modal displacements s_i needed for estimate the equivalent static loads. As it can be argued from Fig. 9a,b the first two modes of the building involves translation in the horizontal directions showing a typical cantilever beam behaviour. The third modal shape is a torsional mode in the vertical direction for the tower. Only after the fourth modal shape it is possible to observe a significant contribution of the other parts of the Castle (Wall C1 & A1). The higher modal shapes are a combination of the transversal vibration mode and the torsional mode for all walls. The distribution of the modal shapes demonstrates that the building displays significant out-of-plane deformations of the walls. Furthermore the plan deformed configurations of the structure confirm that the seismic loads acting along either longitudinal or transversal direction involve remarkable out of plane deformations of the orthogonal structural elements. First two modes activate, respectively, 39.3 % and 44.3 % of total mass.

Table 2 : Seismic load combinations

Load combination	Load coefficient			
	Dead load	Live loads	Seismic load (X-dir.)	Seismic load (Y-dir.)
Comb. 1	1	Ψ_{Ej}	1	0.3
Comb. 2	1	Ψ_{Ej}	0.3	1

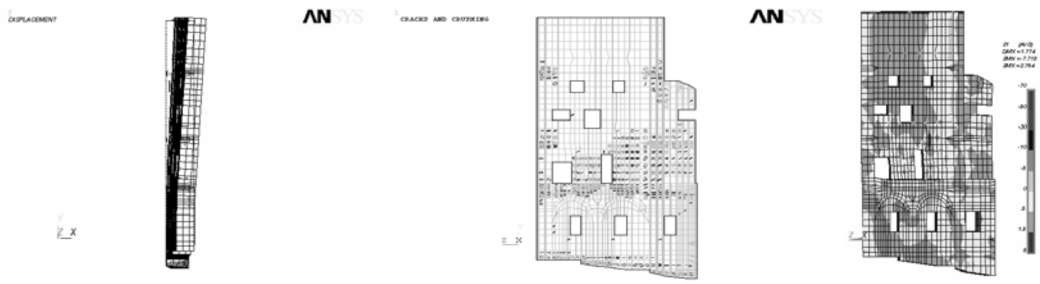
Two loads combinations (Table 2), based on a preliminary static linear analysis are taken into account. For brevity only main results for Comb.1 are next reported in Fig. 10 and Fig. 11.



(a) (b) (c)

Figure 10 : Loads Comb. 1 : (a) Deformed shape ; (b) Principal stress σ_1 [kg/cm^2]; (c) Principal stresses

Under seismic loads the behaviour of the tower is a cantilever behaviour (see Fig. 10a). The Tower, again, pushes against the walls A3 (see Fig. 10a,b) and C2/A2 (see Fig. 11).



(a) (b) (c)

Figure 11 : Loads Comb. 1, Wall A2/C2 : (a) Deformed shape ; (b) Cracking/Crashing path ; (c) Principal stress σ_1 [kg/cm^2]

4 STRENGTHENING DESIGN

The main objective of the strengthening design is to increase the out-of-plane strength of the walls A1 and B1 at the last floor and the load bearing capacity of the foundations. The top of the walls A1 and B1 are linked together through the insertion of lattice trusses in the inclined pitches of the roof. The interior members of these trusses are made of wood beams, with the same dimensions of the existing ones, while the perimeter beams and the diagonals are made of steel. The foundations positioned over the compressible soil are strengthened using vertical micro-piles, whose heads are fixed in two reinforced concrete beams linked to the existing foundations.

5 CONCLUSIONS

In order to assess the structural behaviour, and to evaluate the seismic vulnerability of an Italian Medieval Castle a 3D non-linear numerical model has been created, and then identified in order to take into account the actual cracking path. By some hypothesis concerning ground interaction it has been possible to obtain a proof of the crack pattern, which has been found to depend primarily on the change on ground stiffness. Then the building has been subjected to a seismic static analysis, according to EC8, through the application of horizontal forces in two orthogonal directions not acting simultaneously.

By a comparison between the stresses and the strains produced by the seismic shocks it has been observed that the building damage is mainly due to the movement of the Tower towards the courtyard. The comparison demand (seismic loads) vs. capacity (material and topology strength) confirms the susceptibility of this type of buildings to extensive damage and possibly to collapse. Besides, the efficiency of current techniques for repairing and strengthening have been analyzed in order to evaluate their benefits.

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