Static and dynamic analyses of "Maniace Castle" in Siracusa-Sicily

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ABSTRACT: A case study is presented of preliminary experimental and theoretical investigations on the static and dynamic structural behaviour of a XIII century monument in Sicily, Maniace Castle. The castle was built facing the sea on a square plan. The study is based on in situ investigations and on theoretical analyses calibrated on the results of the tests. The basis of the experimental work has been the characterisation of the local state of stress and the most relevant properties of the materials used. Numerical models, suited to static and dynamic analyses, have therefore been constructed and roughly calibrated. More sophisticated dynamic investigations and more precise calibrations of the numerical models, based on dynamic identification procedures, have then been conducted. The results obtained so far show very peculiar and potentially dangerous aspects of the static and dynamic properties of the structure.

1 INTRODUCTION

1.1 Historical description

The XIII century Maniace Castle (built between 1232 and 1240, named after a famous Bizantine general) is a masterpiece of the military architecture from the time of Emperor Frederick II.

The castle was built facing the sea on the Isle of Ortigia (ancient Syracuse, where the Greeks had founded one of the most important towns in Magna Grecia) on a square plan. The original structure consisted of square enceinte, with round towers at the four corners: these enclosed a perfectly square single hypostyle chamber (named "Salone"), divided into 5 spans per side. The twenty-five spans were set out in a double row around an atrium with an impluvium.

Figure 1: Panoramic view of the Maniace Castle.

Restructuring is documented in the XV century (when it was used as a prison), in the XVI century (within the realignment of the Siracuse city walls by Ferramolino) and in the XVII century. The current state of the castle is a direct consequence of an explosion during the Spanish occupation, on 5 November 1704, that devastated the original structure. The subsequent token renovation included the erection of walls to form stores. The gunports go back to the use of the castle during Napoleonic times, while the blockhouse is from the Bourbon era. Military use of the castle continued up to the Second World War. Of the original structure, the enceinte and the towers remain and the SE section of the hypostyle “Salone”.

The damage and the interventions in relation to its continuous use as a military facility makes it very difficult to correctly interpret its current state of conservation (from the structural point of view) and forecast its behaviour under the expected environmental actions, among which, those caused by earthquakes (it is in an area of very high seismic risk) are of major concern.

2 DIAGNOSTIC INVESTIGATIONS

2.1 Static characterisation

A first series of investigations regarded some structural elements in elevation, acquiring metrical and compositional information by mechanical surveys of continuous core sampling and successive inspection of the boreholes with a colour television probe, determination of the state of stress and the deformability characteristics through tests with flat jacks and a study of texture uniformity by geophysical analyses and radar.

The core sampling made it possible to identify different wall textures belonging to different building periods, ranging from mortar of poor consistence due to the low percentage of binder and very low adherence to tough mortar cohering well with the stone elements. The variability and consistence of the system of voids was clarified using the television probe.

The tests with the flat jack ascertained that, while the north and west towers had similar stress values, 0.31 MPa on average, there are load eccentricities in the east and south towers (original masonry) with average values of around 1 MPa, justifiable on the basis of the original construction techniques and to probable rotations of the towers. Other interesting situations were found in the internal courtyard, where the value registered on the buttress leaning against the perimeter wall of the main structure that supports the weight of the arches forming the vaults, denotes a possible rotation of the masonry towards the courtyard. In the main castle structure, the “Salone” divided in two by a series of columns on which the ribs of the cross vaults that constitute the roofing rest, average values have been registered in the free-standing columns of 3.75 MPa. The stairs and other masonry connecting the towers show low stress values, that are also confirmed in the containing walls of the embankment on the south-west side at sea level.

The tests with double flat jacks were sited in order to characterise the different wall types found. The brickwork edged in stone and bricks have very high deformability values of 11110 and 10000 MPa, the chaotic masonry have low modules varying between 900 and 4800 MPa, while in the filling masonry the values become very low, around 350 MPa. The tests of the masonry in the crown stones, in both the calcareous and lava stone, have very low deformability values of between 850 and 930 MPa, with an estimate of breaking stress at between 1.20 and 1.60 MPa.

Because of important and widespread cracks, non-destructive geophysical inspections were done on two columns situated on the ground floor of the main “Salone”. Cross-hole type measurements (sonic and radar) characterised the columns globally, while horizontal sonic tomography detailed the internal constituent conditions limited to the section examined. These investigations confirmed the continuity of the cracking or lack of cohesion inside the columns, while they excluded the presence of voids or extended cavities.

2.2 *Dynamic characterization*

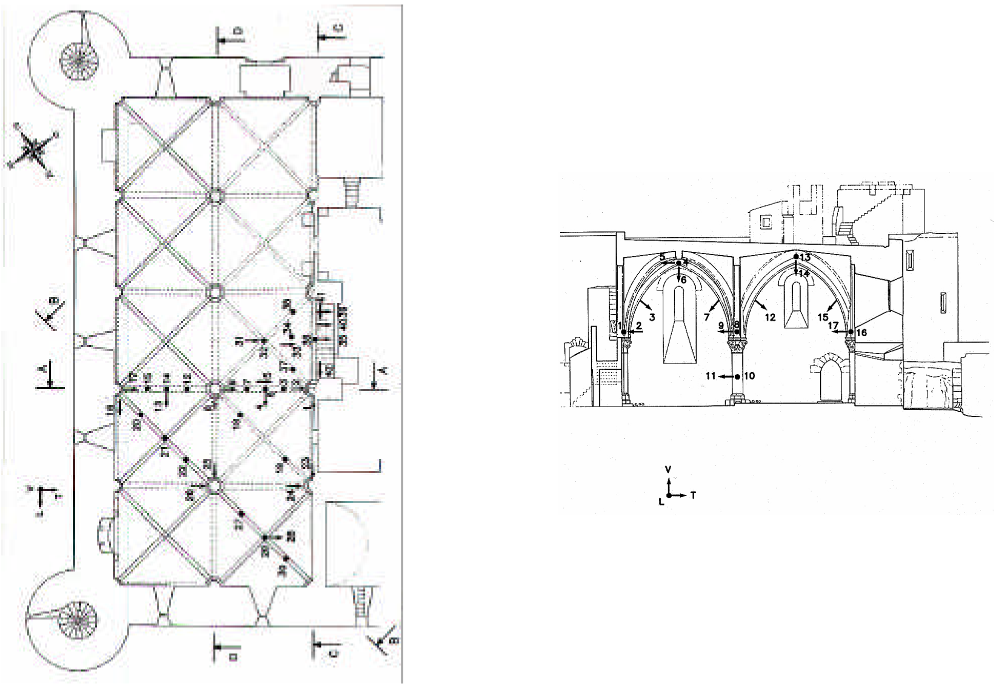

Figure 2: Excitation and measure positions of dynamic tests.

Analysis of the dynamic characteristics of a building is aimed at evaluating the responses of the building to actions imposed that vary over time or natural actions occurring at the time of the registrations.

Three different methods were used in these tests:

- measurement of the response of the structure to the application of a variable sinusoidal force in correspondence to a central column in the “Salone”,
- measurement of the response of the structure to an impulse excitation applied to the “Salone” roof;
- measurement of the response of the structure to environmental actions (wind).

The forced excitation tests were conducted using an electrodynamic exciter capable of generating unidirectional sinusoidal forces of a regulable amplitude and frequency within determined limits, installed in correspondence to a central column in the “Salone”. Three series of tests were done, exciting the column above the capital in longitudinal and transverse directions and at the mid-shaft in a transversal direction. The field of frequency explored was 3-70 Hz with average amplitude of the force of 600 N.

The other tests registered the vibrations caused by environmental actions (wind) or by expressly created events (a platoon of soldiers marching on the roof).

The response of the building was measured using accelerometers set out in order to pick up both the displacements of the whole structure and those between some sections. The measurement network was composed of 42 positions, acquired by repeating the tests twice.

Scaffolding was erected to ease access to the vault and columns to install the exciting and measuring instruments.

Figures 2 show plans of the measuring and exciting positions.

Analysis of the transfer functions of all the measuring positions and all excitations allowed the principal vibrating modes of the complex and those of the central column to be identified.

Based on the FRF, the natural frequencies were identified in correspondence to the major peaks, hypothesising the absence of coupling, while the components were always calculated on the basis of the amplitude of the peak.

The modes of the structure were identified at the 5.1 and 9.5 Hz frequencies in a transverse direction and 12 and 13.7 Hz in a longitudinal direction. The central column frequencies were identified at 37.1 and 38.3 Hz in a transverse and longitudinal direction, respectively. It should be noted that the highest values of the mode shapes are almost always in correspondence to the columns.

The field of frequency of interest was defined as follows:

- the lower limit must be low enough to include the basic frequency of the system (around 5 Hz) and sufficiently high to cut out the noise associated with lower frequencies; 3 Hz was adopted;
- the modal truncation must guarantee an adequate description of the seismic behaviour of the building. In particular, the weight put in motion by a movement at the base in a SE-NW direction must be above 90% of the total weight. Modal analysis of a preliminary FEM model shows that an upper limit of 12 Hz is more than sufficient.

The non-linear optimisation procedure of the experimental FRF allows the modal parameters to be derived that provide the FRF best approximating the experimental curve.

3 COMPARISON BETWEEN THE EXPERIMENTAL RESULTS AND FEM MODELS

The data processing consisted mainly of extracting deformability data to introduce into the numerical models and parameters of "experimental" dynamic behaviour using modal identification techniques. The numerical models were calibrated on the basis of both a direct comparison between the values of theoretical local stress and measured values and the application of more complex techniques of structural identification in the dynamic field. Lastly, structural analyses were done in the static and dynamic field with the calibrated models.

Numerical modelling was done in the linear elastic field, the limits and potentials of which are well known. However, a global model, even if elastic, of a building with very complex geometry is extremely useful for interpreting the overall behaviour and both static and dynamic local reactions found in the experimental investigations and examination of the instability, to highlight local situations of specific interest in relation to the appearance of particularly dangerous stress states, especially where it is very difficult or impossible to do experimental checks.

The model is also useful for guiding the identification and choice of possible collapse mechanisms for further investigations and analyses in the non-linear field.

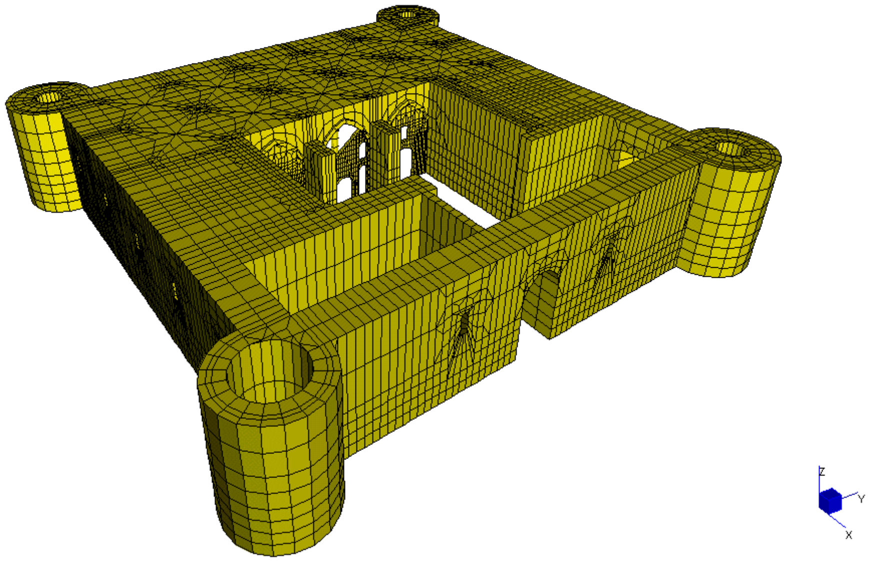

Figure 3: Finite element model of the castle

The spatial (or physical) model of the building was obtained by the finite elements technique (fig. 3). For the modelling, hypotheses were made of elastic-linear behaviour of the materials, admittance (dynamic deformability) of the soil, negligible in the field of frequencies of interest, with the aim of evaluating the global stability of the building and rigid restraints.

Information on the building geometry was acquired by direct on-site measurements, photographic and architectural surveys, while that relating to the materials was supplied by tests with flat jacks and core sampling.

The mesh having been completed, it was possible to go ahead with the different types of analysis:

- static analysis in conditions of linear elastic material;
- dynamic analysis by a seismic spectrum.

4 LINEAR STATIC ANALYSIS

As well as the weight of the building itself, by far the most important load condition in non-seismic situations, the effects of wind were analysed.

Some of the results are summarised in the following table 1. In particular the values at the base of the columns were evaluated: with the permanent load alone they corresponded perfectly with the experimental values obtained.

The results from the tests with the flat jacks are compared with the values of theoretical stress due to the permanent load obtained by the FEM model.

Table 1: Comparison between the FEM theoretical stress values and experimental values.

Test	Experimental stress [MPa]	Theoretical stress [MPa]
1	0.36	0.27
2	0.36	0.34
3	0.27	0.22
4	0.72	0.25
5	0.54	0.61
6	0.09	0.10
7	0.27	0.25
8	0.16	0.17
9	0.00	-
10	0.09	-
11	0.72	-
12	0.27	-
13	1.80	0.30
14	0.27	0.25
15	1.40	1.60
16	0.09	0.07
17	0.10	0.11
18	4.20	4.80
19	1.50	1.80
20	0.08	-
21	0.09	-

5 STRUCTURAL ANALYSES UNDER THE EFFECT OF SEISMIC ACTIONS

The effect of an earthquake was analysed following the procedures indicated under Italian Code. The stresses caused by horizontal or vertical seismic actions must be evaluated conventionally through equivalent static analysis or a more detailed dynamic analysis. It is assumed that ground motion can happen, not contemporarily, in two orthogonal horizontal directions. The weight of the structure undergoing the movement caused by the earthquake are those of weight and permanent overloading, plus a quota of incidental overloading, that in this case is nil.

5.1 *Equivalent static analysis*

Static analysis of the earthquake effects can be done for buildings with a regular structure and with elements of normal span.

The seismic effects can be evaluated conventionally through static analysis of the structures subject to a system of horizontal forces parallel to the directions hypothesised for the earthquake and a system of vertical forces, distributed on the building proportional to the weights.

5.2 *Modal analysis*

The first ten vibration modes (fig. 4) of the castle were examined even if, as shown by the subsequent spectral analysis, just the first five are sufficient in the two directions to obtain a coefficient of modal participation of the weight above the minimum required by Code.

As mentioned previously, there are many modal forms in the field 9-12 Hz (fig. 5).

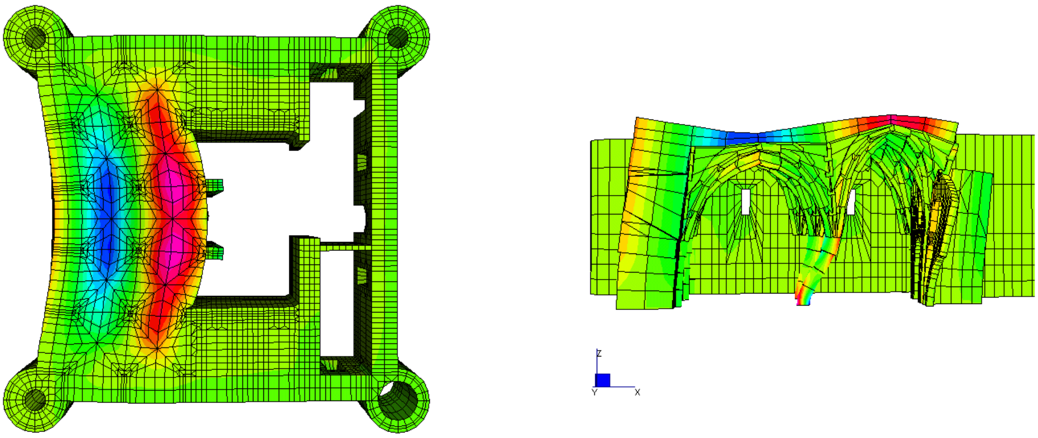

Figure 4: First vibration mode of the structure (f_1=5.4 Hz)

5.3 *Spectral response*

Seismic effects can also be evaluated conventionally through dynamic analysis of the building in the linear elastic field. This can be done using the modal analysis method, adopting the seismic response spectrum from the Italian Code. The method is based on modal superposition. This coincides with the calculation of the modal responses and their combination to determine the maximum response of the building: 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 included in the analysis. The first ten vibration modes are examined, but only the first 5 modes in the two orthogonal directions x and y have been identified as representative of the behaviour of the building: the respective modal weights amount to more than 90% of the total weight for the analyses along the x and y axes, respectively.

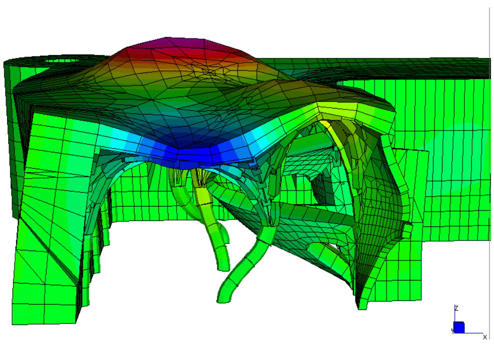

Figure 5: 4th mode of the structure (10.44Hz; section).

Table 2: Modal participation factor (earthquake component along x direction).

Mode	Amplitude	Participation factor
1	1.63E+00	69.47%
2	4.11E-02	1.76%
3	8.34E-03	0.36%
4	4.63E-01	19.76%
5	1.50E-02	0.64%
6	1.21E-02	0.52%
7	2.83E-03	0.12%
8	1.03E-02	0.44%
9	1.42E-01	6.08%
10	1.99E-02	0.85%

6 CONCLUSIONS

6.1 Judgement of the results of the identification

The following conclusions can be drawn from the identification procedure:

- Based on the results of the frequency response functions supplied by the tests, the modal parameters relating to the first 5 system modes were extracted. The first frequency is around 5.2 Hz and concerns the SE wall.
- Identification of the models on the basis of the experimental mode shapes led to the identification of weight density values of the materials compatible with those deduced from the local surveys and visual analysis, and of weight distribution compatible with the surveys and measurements on site.
- The experimental modal shapes are also compatible with the hypothesis of soil with marginal dynamic deformability, at least in the case of the first frequencies.
- Identification of the models on the basis of the values of experimental natural frequency led to the calculation of the values of modules of equivalent elasticity varying between 300 and 10000 MPa. They coincide, within the limits of the geometrical precision assumed, with the values realistically expected and are confirmed by the results of the local tests (core sampling, flat jacks).
- A large number of modal shapes in the field 9-12 Hz was confirmed, as suggested by the preliminary FEM model and confirmed by the definitive model. The modes result strongly coupled and, as already mentioned, concerns the entire hypostyle “Salone”.

6.2 Stress state

The action to which reference must be made to evaluate the safety conditions of the construction in terms of local stresses acting on the materials (i.e. under all conditions except an earthquake) is undoubtedly the weight. The contribution of the variable actions (wind was examined) is irrelevant, both as instantaneous effects and, even more so, as effects extended over time (more insidious).

The situation regarding seismic action is naturally more complex, as it is unrealistic that “real” effects can be evaluated in purely theoretical terms, taking into account the characteristics of the building. The previously-mentioned conventional analysis methods can provide (taking into account the limitations of the available data, especially as regards the survey and definition of the construction and mechanical characteristics of the vaulted system) important qualitative information for indicating the evolutionary trends of stress states present in static situations and identifying possible global behaviour that could be dangerous to the structural integrity, and therefore highlight the necessity and steer the choice of possible interventions.

Within this context, the experimental tests and numerical analyses point out signs of real critical conditions with regards to the safety conditions of the columns.

The direct comparison between the experimental measurement and theoretical evaluation is possible for the two external columns, where high axial loads and their marked eccentricity (that can be explained, other than by possible and very probable lack of homogeneity, by past interventions, etc.) in a precise structural behaviour demonstrated by the numerical modelling, involve values of compression stress of around 5 MPa. The numerical analyses also confirm similar values for the central columns.

This situation requires attention, taking into account that:

- the few data obtained in the laboratory tests on stone samples, presumably of the same type as those used to build the columns, indicate resistance values that can be very high, but also extremely variable. The lowest value (in order of amplitude) is just 10 MPa;
- the resistance of the ashlar masonry that the columns were built of is clearly lower than that of the stone elements;
- seismic action on the columns entails, as an effect of a structural behaviour that will be commented on in the following section, a substantial increase in the eccentricity of the normal stress, and a corresponding aggravating of the state of stress that the numerical analyses estimate as a significant partialisation and an increase in the compression stress in the order of 40% (therefore bringing the maximum stress very close to the minimum value of resistance obtained in the tests on samples of stone).
- the non-destructive tests indicate the existence of cracks (and/or voids or cavities) deep inside the most stressed parts of the columns, where lesions have also appeared on the surface.

In the rest of the structure, states of stress are verified that are in general not worrying, with concentrations of stress sometimes traceable to easily identifiable local behaviour (arrises, buttresses, etc.), sometimes not shown by the numerical analyses (e.g. in the south tower) and presumably connected to the effects of the explosion that partly destroyed the castle.

These latter can be taken into account at the stage of treating the masonry, maybe by increasing the level of the localised improvement operations (e.g. injections, limited reconstruction).

The columns are more problematic. In the following section comments will be made on the aspects connected to the possible effects of seismic action, but independently of any work to improve the structural behaviour of the monument as a whole. It is obvious that the possible hypotheses of specific reinforcement of the columns in relation to the stresses present under static loads must be seriously considered.

6.3 Dynamic behaviour

The most significant aspect of the investigations in the dynamic field, well demonstrated by both the experimental tests and numerical modelling, is that besides the first mode (that involves symmetrical flexural distortions of the masonry structure and that, as expected, gives the most important contribution to the dynamic response), the contribution of a very complex coupled mode, no. 4 (fig. 5), is extremely important, that involves a flexural-torsional deformation of the vaulted system and accentuated flexural deformations of the columns. Others with similar characteristics exist in a very narrow field of frequency, a fact that confirms the significance of the phenomenon independently of the limits of both the experimental investigations (number and position of the instruments, type of forcing action used) and of the numerical analyses (regarding the significance of the information available).

Further study of the construction and dimensional characteristics of the vaults therefore appears to be inevitable: the possibility of identifying operations that would modify the contribution of the dynamic response thus highlighted could lead to significant improvements. Regarding this, it is enough to note (even although it is not the only factor of interest: the motion component shows a structural “irregularity” anyway dangerous in an earthquake) how mode no. 4 influences the flexural deformation of the columns, causing a substantial worsening of the stress state in the terms discussed in the previous section.

REFERENCES

Bonzi A., Eusebio M., Pizzigalli E., Ravasio F, 1997. Dynamic tests and monitoring of the Leaning Tower of Pisa: aims, methodology and results. In *IABSE International Colloquium, Seriate (BG) Italy, Proceedings,* p. 153-160.

Chopra, A.K. 2000. *Dynamics of Structures.* Upper Saddle River, New Jersey: Prentice Hall.

Ciurnelli F., Macchi S., Ravasio F., 1997. Sonic tomography and radar investigation on the pillars of

Pavia Cathedral. In *IABSE International Colloquium, Seriate (BG) Italy, Proceedings*, p. 207-214.

Clough R.W., Penzien J. 1993. *Dynamics of Structures.* Singapore: McGraw Hill.

Ewins D.J. 2000, *Modal Testing, Theory, Practice and Application.* Baldock, Hertfordshire, England: Research Studies Press.

Halvorsen W.G., Brown D.L., 1977, Impulse Technique for Structural Frequency Response Testing, *Sound and Vibration*, November.

Menga R., Pizzigalli E., Ravasio F. 1998. Seismic monitoring system of the San Francesco Basilica in Assisi – Italy, In *Workshop on Seismic Performance of Monuments, Lisbon 12-14 November, Proceedings*, p. 57-68.

Bendat J.S., Piersol A.G., 1982. *Engineering Application of Correlation and Spectral Analysis.* New York: John Wiley and Sons.

Pizzigalli E., Ravasio F., Mazza D., Terzi E., Di Lullo S. 1997. Static and dynamic characterization of Gombito Tower in Bergamo. In *IABSE International Colloquium, Seriate (BG) Italy, Proceedings,* p. 169-176