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DIAGNOSTIC TESTS vs STRUCTURAL MODELS: THE UTILITY OF THE COMPARISON

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Abstract

Static and seismic assessment of historical buildings requires the knowledge of geometrical dimensions, mechanical behaviour and stress status of the relevant structural elements. These data allow implementing structural models reliable to predict the actual behaviour of the buildings.

This paper presents the case history of three historical buildings, Besta Palace, Besta Manor and Masegra Castle, located in Valtellina, an alpine valley in the northern part of Italy. They were built in the Renaissance period with the techniques typical of this area: chaotic stone masonry built using local metamorphic rocks, high thickness masonry vaults, wooden ceilings and tile roof overlays.

The research program planned to subject these buildings to a complete set of mechanical characterization tests: laser-scanner surveys, crack and deformation monitoring, radar surveys, borehole inspections, flat jack tests, vault tie rods vibration measurements and wood density measurements. The individuation of recurring morphological characteristics allowed to reduce the number of the required tests and to extend the results to similar situations within the analyzed buildings.

Both F.E.M. and limit analysis models were implemented to assess the static and seismic safety; accuracy of the models was then assessed comparing the model output with the experimental data. The calculated masonry state of stress and tie rods tensile stress status were compared with experimental data to allow improving the precision of the model. Similar comparisons were carried out between the actual and calculated displacements and deformations. This procedure, performed with typical trial-and-error method of structural models implementation, allowed evaluating how the models predict the actual structural behaviour of the buildings.

The results are relevant both for the extension of the diagnostic tests and for the method applied to assess the models though the use of the experimental stress and strain data.

Key words

tests, models, heritage

Introduction

By now it is accepted in the profession that the understanding of the structural behaviour of an existing building must be based on knowledge, as accurate as possible, of its geometric and construction characteristics, its conservation, and its materials. It can be achieved only through an appropriate survey campaign and experimental investigation that allow the gathering of all the data required to implement a model that properly represents the structural behaviour of the building. Such a model is necessary to have analytic data on which to base a proper assessment of the safety of the building in static and seismic conditions, but also to simulate the effectiveness of the designed interventions of enhancement or improvement of the structural characteristics.

In order for the model to properly represent the real behaviour of the structure, it is necessary to perform its validation not only with regard to the adequacy of the numerical calculation code, but also with respect to its accuracy in representing the actual structural behaviour. It is necessary at this stage to have experimental data provided by the tests carried out on site, which should be compared with the numerical results provided from the model. Only a comparison can confirm the adequacy of the implemented model and assure the validity of the obtained results. The present paper describes a research program carried out in the Province of Sondrio (Lombardy – Italy) and developed by the authors for the structural aspects. The project involved several public and private buildings located in Valtellina and has been developed in coordination with the main entities involved in the management and conservation of cultural heritage; the Milan Office of Government Department for Architectural and Landscape Heritage, public authorities, religious organizations, foundations and associations (1).

This project aimed at developing techniques for preventive and planned conservation (2) involving a few significant historical buildings, and starting with an in depth understanding of their characteristics: structural surveys, materials analysis, thermo-hygrometric measurements and structural, environmental and surfaces conservation monitoring (3) (4).

1.1 Description of the analyzed buildings

The buildings are located just a few kilometres away from each other and are representative of one of the periods of greatest economic, cultural and artistic improvement of the Valtellina region, the period from 1512 to 1797 in which, because of its geographical position, the valley was under the government of the state of the “Three Leagues” (the actual Canton of Graubünden in Switzerland) and therefore provided an easier travel connection between the domains of the Habsburgs in Italy (Duchy of Milan) and in the Empire (Austria). Consequently this territory played a political and military role absolutely relevant to the historical context of the time, especially during the events of the Thirty Years' War.

The present paper aims to analyse three buildings: Masegra Castle (M.C.) in Sondrio, Besta Palace (B.P.) in Teglio and Besta Manor (B.M.) in Bianzone. In their present conformation, each building is evidence to construction phases mainly carried out in the sixteenth-seventeenth century, although they also show traces of several earlier stages.

The three buildings, which are currently under public ownership, have very different features and conservation status. The Masegra Castle was the seat of the Military District until, in 2004, its use was finally granted to the municipality of Sondrio. Currently less than half of the building has been restored and it is used as a museum; the remaining part is abandoned, although it does not present worrisome signs of instability.

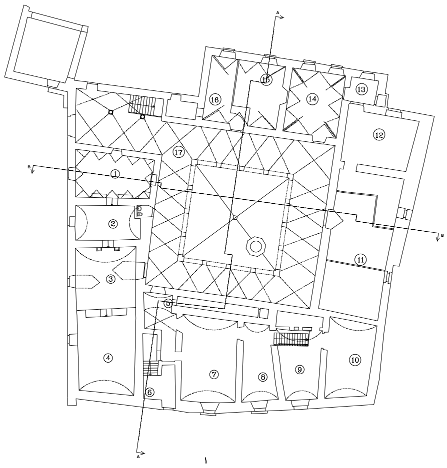 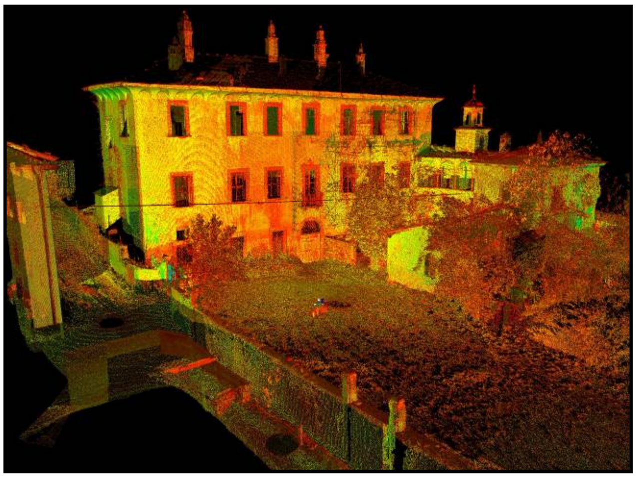

Figure 1 P.B.: plant – B.M.: point cloud of laser-scanner survey

The Besta Palace, privately owned in the past, has been acquired by the Italian state in several stages between the late nineteenth and early twentieth century (5). Later, following a long period of restoration work undertaken in 1912, it has been used entirely as a museum, managed directly by the Milan Office of Government Department for Architectural and Landscape Heritage. The Besta Manor, a private estate until 2003, was left in a state of complete abandonment. When it was later acquired by the municipality of Bianzone, its state of preservation, based on the first results of the analysis carried out for this project, was extremely critical. The immediate structural danger resulted in emergency reinforcement by provisional shoring to prevent collapse of the most unstable parts of the manufacture. It is noted, incidentally, that none of the major phenomena of structural damage found in the tested buildings were due to seismic origin.

The research program is of significant local importance in the territory of the Province of Sondrio, since the homogeneity of the historical context has produced a remarkable homogeneity and permanence of construction techniques. In this context, the development of an investigative methodology of some buildings and above all, the collecting of diagnostic data, provide results that can be applied to a wider context that until now, has been little studied from the point of view of building techniques. In addition, the project has is valuable in a wider context, since the analyzed masonry types (random rubble masonry consisting of strong rocks and weak mortar) as well as the analyzed building types (very thick stone masonry vaults, wooden ceilings, wooden roof trusses) are of dominant recurrence in the entire Alpine region. Therefore, the proposed methods and the data collected are widely applicable not only in the national context, but also in a transnational one, an effort that has already been undertaken in Switzerland as part of a joint research.

1.2 Description of the analysed masonry

The typical masonry of the Alpine area can be described as coursed rubble masonry, varying in size, with a predominance of horizontal rows. In section, these walls, which have, in the ground floor of the analyzed buildings, thicknesses ranging from 60 to 100 cm, are constituted by two poorly or disconnected wythes. The inner core is made of the same material, in this case selected with smaller and irregular size and placed in a disorderly manner. This masonry is generally made from rather strong stone (in relation to the geology of the area it is generally crystalline rock in the western Alps, including our study area, and dolomitic and calcareous in the eastern Alps) and lime mortar, mechanically weak, with aggregate of crystalline or calcareous type, in accordance with the two geographic zones defined above.

This type of wall generally is provided with poor clamping in the corners of the buildings and it is hardly suitable to give the building a three-dimensional structural behaviour. Historically, to provide systematic connections in these walls, elements with the function of tie rods, especially wooden elements embedded within the masonry, were clamped at the ends with iron anchor plates. These elements were usually weak for both the frequent degradation of the wood (that deteriorates because it is embedded within the masonry which often contains a high level of humidity) and for the systematic failure of the wood. In addition, the iron joints often were undersized, degraded and inefficient.

1.3 The path of knowledge

The current Italian structural set of standards concerning cultural heritage (6) (7), points out the fact that is still not possible (or desirable) to achieve the complete knowledge of a building from a structural point of view. For this reason, the standards trace a path of knowledge that can be pursued with different levels of detail, depending on the accuracy with which the operative methods such as survey, historical research and experimental investigations are conducted. The investigative process is aimed at the definition of an interpretative model of the structure which allows both qualitative interpretation of the structural behaviour and quantitative evaluation consequent to the execution of an actual structural analysis. This focuses attention, in particular, on the fact that it is necessary in any case to understand the behaviour of the building, to compare it with on-site observations and still to be able to give an intuitive explanation of the expected phenomena. Otherwise the results could be mere arithmetic calculations not fully representative of the real physical behaviour of the building.

To completely analyze the construction, it should therefore draw attention to the need to reach an acceptable level of understanding of the building that, in our context, should be developed according to the following activities:

- survey of the building, cracks and deformation patterns;
- interpretation of the historical evolution of the construction;
- structural identification of the building and of its construction details;
- evaluation of mechanical properties of materials and their deterioration;
- evaluation of soil-foundation relationship.

It's appropriate to emphasize the fact that the knowledge of the building resulting from the activities of survey and diagnosis cannot be regarded without a proper understanding of the historical evolution of the construction. As a matter of evidence, the mere fact that a building has so far remained stable enduring differential settlement, winds and earthquakes, constitutes in itself proof that the building is adequate to ensure long-term stability. It can be assumed that time and history has set up a full-scale experiment on the building and that the cracks, distortions and states of stress represent its results. It is important to measure them in order to obtain a model that comes as close as possible to the real and proper structural behaviour of the construction.

Experimental and numerical analysis is a powerful tool to corroborate empirical data making more objective the historical observation. However, dealing with existing buildings, the current analysis tools, even the most advanced, are often inadequate to explain the stability of complex structures that have proved their endurance over time.

2. Experimental investigations

As stated above, the evaluation of the mechanical characteristics of the materials constitutes only one of the steps on the path of understanding. The Italian structural standards listed above explicitly state that non-destructive techniques (indirect ones) make it only possible to assess the homogeneity of the mechanical parameters in different parts of the building, but they do not provide a reliable quantitative estimation of the relevant mechanical characteristics since they concern themselves with the measurement of indirect parameters (such as e.g. the pulse velocity). Direct measurement of the mechanical parameters of the masonry, in particular the resistance ones, can be performed only through slightly destructive or non-destructive surveys. Calibrations of non-destructive techniques with destructive tests can be useful to reduce the number and the invasiveness of the investigation.

The analyzed buildings were subjected to a careful preliminary inspection that allowed for the identification of the most significant deformation and cracking phenomena and to design the campaign of diagnostic investigations aimed at the analysis of the material and structural aspects.

Figure 2. (B.P.) - Non-destructive testing and slightly destructive testing

The campaign was carried out according to the following guidelines:

- the overall geometric survey of the buildings was performed with laser-scanner techniques, allowing for the identification of structurally significant items and the relief of the crack pattern and framework deformation;
- the structural survey of the construction was performed by inspecting the elevation walls by horizontal drillings and video-endoscopy; the characteristics of vaulted structures were analyzed by carrying out non-destructive measurements with geo-radar and making small exploratory samples aimed at measuring the thickness of the vaults, of the upper filling and of the flooring superposed; the stress state of the tie rods was detected by vibration measurements, and the characteristics of the wooden floors were determined by making samples and endoscopic inspections;

- the mechanical properties of the masonry were assessed through tests with flat jacks to determine the state of stress and the characteristics of deformability (see figure 2);
- the properties of wooden trusses and their state of deterioration have been characterized by penetrometer tests carried out with Resistograph;
- to analyze the foundational structures, sub-vertical drillings associated with video-endoscopy survey were conducted, and aimed to identify the geometry and composition of the foundation structures; in some cases were performed samples for direct prospection, geophysical and geotechnical investigations to determine the characteristics of the foundation soils.

As suggested by the standards, the number of slightly destructive tests have been limited to a minimum, the information collected on the three buildings has been cross-referenced when possible.

The tests carried out in different contexts have been listed below in table 1 where the following indications are specified: °=activity performed by Milan Politecnico; °°=activity performed by CNR – Italian National Research Council):

Table 1 - experimental activities carried out on the three buildings

Investigated elements	**Experimental investigation**	**Masegra castle**	**Besta palace**	**Besta manor**
Soil	Geotechnical characterization	no	yes	no
Whole building	Laser scanner survey	complete°	partial	complete
Vaults	Radar survey	no	yes	yes
Vaults chain	Tie rods vibration measure.	yes	yes	yes
Masonry	Boreholes survey	yes	in progress	yes
Masonry	Single flat jack test	yes	in progress	yes
Masonry	Two flat jack test	yes	in progress	yes
Whole building	Structural modelling	yes	yes	yes
Wooden trusses	Resistograph test	roof	ceilings	none
Whole building	Structural monitoring	no	yes	yes
Environment	Thermo-hygrometric measur.	yes°	yes°	yes°
Environment	Thermo-hygrometric monitoring	yes°	yes°	yes°
Mortar & plasters	Chemical-physical analysis	yes°°	yes°°	yes°°

3. Structural modelling

The modelling of a structure consists in its schematization according to a set of relationships that can be expressed with mathematical tools. For ancient structures, it is well-established consideration coming from the observation of the damage caused by the many earthquakes that occurred in Italy in the last decades, and that the actual response of an existing building to seismic actions in many cases may not be determined only by considering the global behaviour of the structure. In fact, it is necessary to take into account especially the response of the individual structural elements (called macro-elements) that are by themselves able to justify the phenomena of local damage systematically detected after earthquakes.

It is therefore necessary to use in a complementary way models submitted to limited analysis, that are suitable to represent structural macro-elements and that well represent the seismic response experimentally observed on structures. The results from the limit analysis models usually are easy to understand and to compare with the real building behaviour. In a complementary way, the finite element method (F.E.M.) models can take into account the overall behaviour of the structure, but the results must be carefully calibrated since they are particularly sensitive to changes in the calculation parameters (e.g. the characteristics of the walls are usually assumed uniform throughout the whole structure, while in reality they can obviously present very significant changes from wall to wall).

A powerful tool on which to base the geometric modelling is definitely the laser-scanner survey that allows for automated algorithms from the point cloud model to the mathematical calculation model (either one, two or three-dimensional) representing exactly the geometry of the structure, and also taking into account detected deformations. Within the research program, F.E.M. three-dimensional structures have been developed for the whole building and for the macro-elements, as well as two-dimensional models that have been analyzed by the finite element analysis or by limit analysis.

3.1. Limit Analysis

The macro-elements can be defined on the basis of the observed damage and disconnections, both of static and seismic origin. Following this criteria, in the case study, the structures were then analytically taken apart into sub-structures; the most important among them were analyzed by kinematic limit analysis (see figure 3).

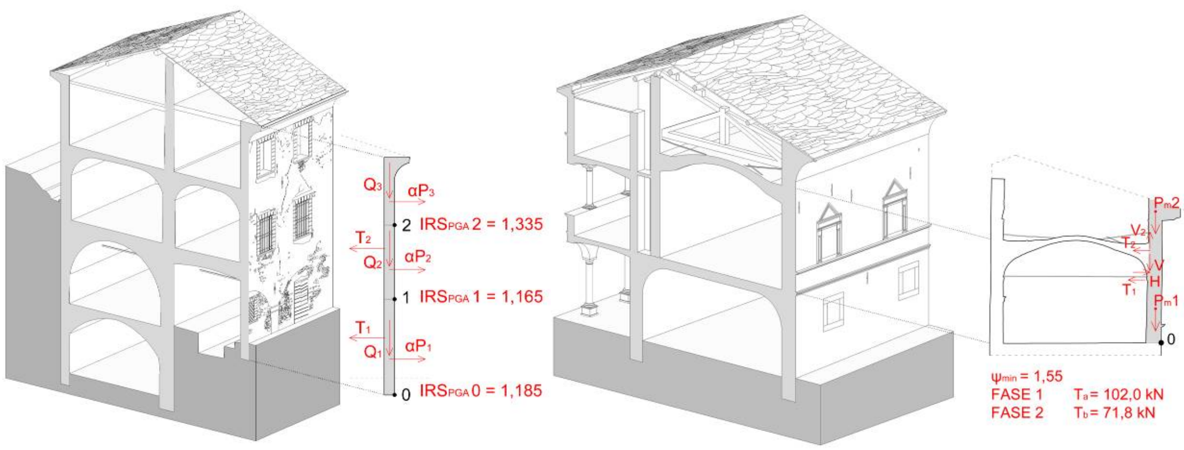

Figure 3. (B.M. e B.P.) Perspective sketch and identification of macro-elements

In Besta Manor, it was highlighted the particular sensitivity of the main facade. The façade is subject to the lateral forces of the vault of the main hall, acting perpendicular to the load bearing wall; these forces are not picked up by tie rods. The situation has been analyzed both in relation with the historical status quo and after the recent intervention that foresaw the placement of provisional tie rods, thus highlighting the effectiveness of the realized intervention.

In Besta Palace the structure of the main hall, which for constructive reasons is structurally independent from the other part of the building, was analyzed by modelling both the vault and the load bearing walls. It was possible to simulate the original situation (with only upper tie rods) and the consequent interventions of the nineteenth century (new tie rods to the vault springers) thus showing also analytically the fact that the upper tie rods were clearly undersized. This justifies the considerable deformation measured on the walls of the façade corresponding to the springers, where the original lateral force of the vault was applied. During the structural enhancement carried out more than a century ago, the remedy to prevent this deformation was the inclusion of the tie rods placed at the height of the springers. The numerical values of tensile stress calculated for the tie rods were used for the comparison carried out at the next pt. 4.3.

3.2. Finite Element Method Analysis

On the basis of the geometrical and mechanical data collected, a comprehensive numerical model of each building that represents the structural elements in space was prepared. The geometry of the buildings has been directly derived from the point cloud model detected with laser-scanner. The walls were represented through shell/plate two-dimensional finite elements, while the elements in stone or wood (e.g. columns, beams, trusses) were represented by one-dimensional finite elements. The mechanical characteristics of the walls were the same as those reported in tab. 2.

The stress values were determined as a first step with the assumption of linear elastic behaviour for the materials, and considering all the walls as perfectly connected and collaborating. This analysis helped to identify the areas in which the tensile stress exceeded the capacity of the masonry tensile strength (which of course is very low). These areas have been put in relation with the location of the detected cracks, highlighting a good coincidence between the results of the calculation and the actual behaviour of the building. Along the lines where the model showed the highest values of traction and simultaneuosly the crack patterns highlighted the presence of disconnections, the elements were disconnected, in order to prevent the transmission of tensile stresses in the model and thus preventing the mutual cooperation between the walls. This procedure was carried out in an iterative way (step by step) coming to a reasonable approximation of the most significant cracking phenomena detected on the structures and to the decrease of calculated stresses because of the resulting redistribution of stress states. This first level of comparison (the observed crack pattern vs. calculated tensile stresses), even if done only in qualitative way, can provide an essential first step of refinement of the structural behaviour as determined by the model, improving the accuracy of the derived results in terms of stress.

4. Model assessment

4.1. Detected vs. calculated deformation pattern

A next level of comparison was implemented by matching the numerical deformation data provided by the E.F. model with those obtained by measurements carried out on site. In this way it was possible to evaluate the influence of geometrical and mechanical parameters conditioning the results of numerical analysis and consequently, the reliability of the implemented models. Concerning the Besta Manor, point displacements detected from the survey (particularly those of the main facade that are the most relevant) were compared with the deformed static pattern calculated by the model, according to the Serviceability Limit Status conditions (SLS - characterized by amplifying coefficients of loads equal to 1). The displacement values of the south facade in some sample points are reported in Table 2. The displacements of the sample points calculated by the model are congruent with what was measured on site; however, the calculated values are significantly different from those measured. In this case, therefore, the approximation of the model is only qualitative, while, in absolute terms, the deformation determined by the numerical model does not come close to the actual detected deformations. Certainly the real deformation may have been amplified by effects such as the long-term deformation of the masonry, the plastic yielding of the ground or cyclic loads applied to the structure; because of these effects, that develop over the centuries, small residual incremental increases produce a accumulation over the long term.

Table 2. B.M. displacement values

SECTION	measured s_x [cm]	calculated s_x [cm]
Sec. B-B	82.8 mm	30.0 mm
Sec. C-C	29.6 mm	25.5 mm
Sec. D-D	00.0 mm	-3.3 mm

4.2. Measured vs calculated status of stress

The comparison has also focused on the status of stress values: the experimental data obtained with the flat jack are compared with those obtained with the E.F. model, once again according to the Operating Limit Status. The calculated status of stress at the base of the outer walls is rather uniform, confirming that, in the lower part of the bearing walls, there is an adequate redistribution of stresses coming from the above structures. The flat jack tests carried out in correspondence with the intermediate bearing wall show an average value very close to the value provided by the model. The experimental difference of the values between internal and external sides of the same wall can be imputed to the non-homogeneity of the masonry, which is, however, evident when considering the extremely high modulus values provided by some of the tests. Similarly, the experimental value recorded on the other bearing walls is significantly greater than the one obtained from the numerical model. Even in this case, the stress pattern provided by the F.E. model is still congruent with the one coming from the on-site tests.

Table 3. B.M. masonry status of stress

TEST nr.	SITE	σv Flat jack	σv model
average M1-2	Intermediate bearing wall (medium value)	0.57 MPa	0.56 MPa
M3	North bearing wall - internal side	0.85 MPa	0,56 MPa
M4	West bearing wall - external side	0.55 MPa	0,18 MPa

4.3. Measured vs. calculated tie rods tensile stress

The analysis of the vaults, considered as macro-elements, was accomplished with different criteria in order to allow for the evaluation of the effectiveness and limitations of modelling by (a) 3-D linear elastic analysis, (b) 2-D linear elastic analysis, (c) kinematic limit analysis (8). It is useful to observe that, also in this case, the models were created by interpolating the three-dimensional point cloud model provided by laser-scanner survey (Figure 4). In the case of the 3-D E.F. model, the number of points of the cloud was first reduced to attain afterwards to their schematization through shell/plate elements; in the case of the 2-D model, the modelling was performed in a similar way starting from the sectional representation coming from the point cloud model. The obtained results were compared with one another and with the results provided by the site investigation and taking into account the usefulness of comparative considerations on the applicability of these criteria. Substantially, the analysis methods provide very homogeneous values for the vertical component of the thrust transmitted to the piers (shifting ± 2% compared with the limit analysis) and values adequately homogeneous for the horizontal component of the thrust (shifting ± 12%).

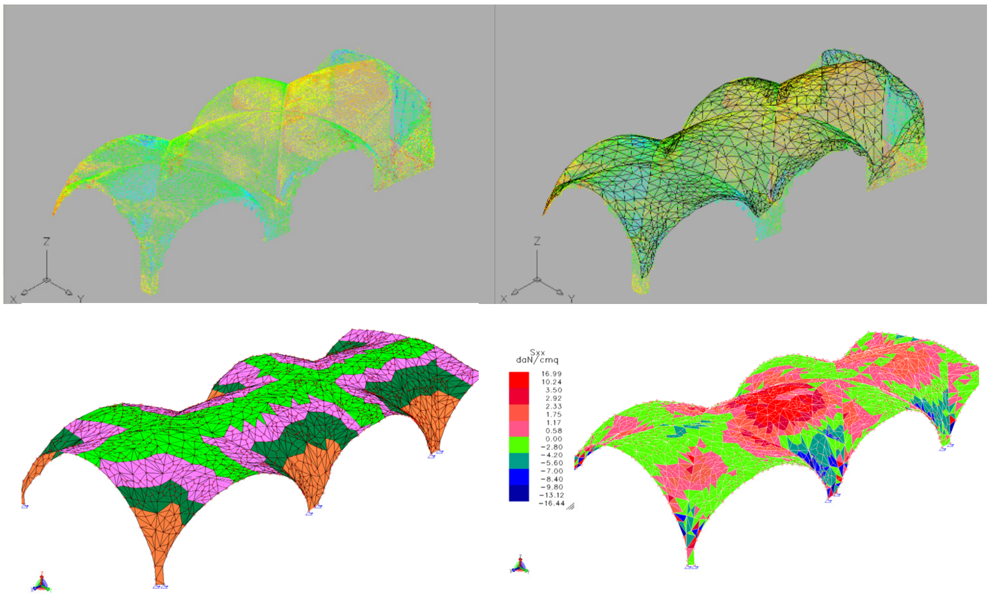

Figure 4. B.M. Modelling and calculation of a vault from the laser-scanner relief

In this case, it was possible to compare the tensile stress values measured by vibration measurements on the tie rods with the horizontal thrust calculated for the vaults. For the vault of the main hall on the first floor of the Palazzo Besta, where there are three tie rods at the springers, the comparison provides very low shifting values between data calculated by limit analysis and data determined by on-site measurements. Overall, it was therefore possible to note that the kinematic limit analysis is the most reliable method both for determining the status of tensile stress on the tie rods and for carrying out vaults structural assessment (9). This analysis does not allow calculating the settlements of an arch's keystone; however even when they are determined by the other described methods, the results are still very far from on-site measurements.

Table 4. B.P. tie rods tensile stress

TEST nr.	H 3-D model	H limit anal (Ψ=1)	H on site meas.
chain E	73,02 kN	71,80 kN	73,40 kN
chain F	60,21 kN	71,80 kN	73,50 kN
chain G	32,13 kN	50,60 kN	50,60 kN

5. Other considerations

This paper does not include considerations dealing with the subsequent assessment of structural safety in the static and seismic fields. It should be noted, however, that the Italian structural set of standards (7) explicitly states that when dealing with historic buildings, it is appropriate to be conscious of the acceptance of a level of seismic risk higher than that of ordinary structures, rather than to use strengthening methods contrary with the accepted criteria of conservation of cultural heritage. In any case, it explicitly requires that a minor nominal life means accepting to provide an additional verification within that limit, as well as to provide a suitable monitoring program. To accomplish this and to control the time evolution of the deformation and cracking phenomena in Besta Palace and Besta Manor, monitoring systems consisting of gauges placed across the cracks and wire gauges connecting separate walls were installed. Data acquisition is managed by a control excitation unit which, at specified intervals (e.g. monthly), is connected to electrical plugs located in an easily accessible position. This allows for the acquisition of data in a discontinuous way (see figure 6), but is also suitable to detect long-term displacements.

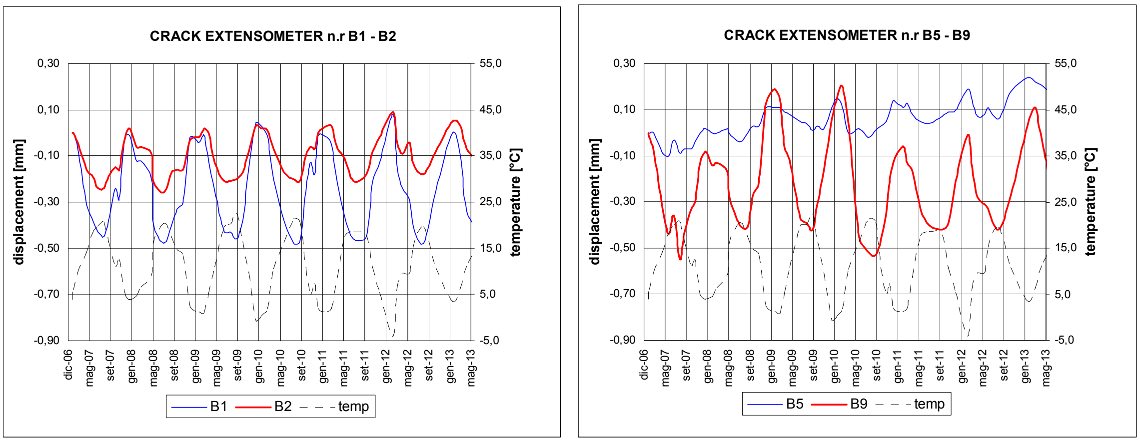

Figure 6. (B.P.) Diagrams of structural monitoring 2007-2013

The monitoring, which in reference to Palazzo Besta now covers a period of more than five years, has provided the necessary data to analyze the evolution of deformation and cracking phenomena highlighting the close relationship between the structural movements and temperature variations; the statistical elaboration also allows discriminating any residual trend. To date, the acquired data have confirmed the validity and the effectiveness of the provisional interventions carried out at Besta .Manor; in fact, the monitoring doesn't detect any long-term trend that could lead one to assume further dangerous evolution of structural movements that have already been observed.

6. Conclusions

The case studies have highlighted the importance of achieving a detailed understanding of the buildings through the application of a well-designed investigation campaign. The acquisition of the diagnostical data has provided the necessary information to study the structures with different computational models, characterized by an increasing level of complexity, by analyzing both the individual macro-elements and the overall building as a whole. The paper has particularly pointed out how, for a correct prediction of the real behaviour of the building, it is essential to verify the adequacy of the structural modelling through a comparison of the calculated numerical values with the experimental data obtained from on-site investigation.

The overall assessments allowed for planning of the execution of specific provisional or definitive reinforcement operations, chosen in accordance with the available economic resources as well as the installation and maintenance of proper monitoring systems aimed to detect the time evolution of the cracking and deformation phenomena that are indication of structural danger.

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