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A Multilevel Approach for the Cultural Heritage Vulnerability and Strengthening: Application to the Melfi Castle

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Abstract: This study outlines a procedure for the seismic safety evaluation of historical buildings for engineers and architects that commonly work on buildings belonging to cultural and architectural heritage. The procedure is characterized by two interrelated phases: (a) building knowledge acquisition and (b) structural behavior analysis and safety assessment. The seismic safety evaluation strongly depends on the first phase, whose data can be obtained according to a multi-disciplinary approach based on five steps: (1) critical-historical analysis; (2) photographic documentation and geometrical survey; (3) structural identification and material survey; (4) foundation and soil survey; and (5) cracking pattern and structural integrity analysis. The proposed method was applied to the evaluation of the seismic safety of the Castle of Melfi (PZ, Italy). Comprehensive and multi-disciplinary knowledge of this monument greatly facilitated an accurate seismic analysis of this monument, which was conducted both at a local and global level using a linear kinematic analysis and non-linear static (pushover) analysis, respectively.

Keywords: heritage structures; building knowledge; critical-historical analysis; pushover analysis; cracking pattern

1. Introduction

The seismic safety evaluation of existing masonry buildings is a challenging issue, whose difficulty depends on the large variety of existing structural typologies that do not allow the formulation of a single and reliable strategy for their structural analysis. For existing masonry buildings belonging to cultural and historical heritage, the challenge is intensified by the conflict between the conservation requirements and the need to safeguard their structural safety, more specifically when the building hosts strategic functions that involve public functions and the presence of large crowds. Moreover, there are objective difficulties in performing tests on materials due to the need to preserve the cultural and artistic value of monumental structures. Currently, conservation agencies (such as the Mibact—Ministry for Cultural Heritage—in Italy) do not easily grant permission for destructive or partially destructive tests and engineers can only rely on non-destructive investigations such as visual analyses, video endoscopy of cavities, thermo-photography and chemical analysis of mortars, among others [1–4]. Finally, very few technical standards exist for heritage masonry structures [5–7]. A major issue is the identification of an appropriate static scheme for buildings that have been modified several times during their history, as discussed by [8].

Every existing monumental masonry building is characterized by a specific and unique construction history consisting of an initial body modified over centuries through substitutions, deletions or additions. This requires that the seismic safety evaluation of monumental buildings is performed according to a multi-stage approach based on two interrelated phases: (a) building knowledge acquisition and (b) structural behavior analysis and safety assessment. The seismic safety evaluation strongly depends on the first step, which gathers all information on the history of the construction, its evolutions, its geometry, the characteristics of the masonry texture and of the soil/foundation system. The proposed procedure was applied within a project funded by the Society for the Development of Art, Culture and Performance Arcus Spa and the Italian Ministry for Cultural Heritage and Activities and Tourism (MiBACT). MiBACT selected several museum structures hosted in historical buildings in order to assemble a set of examples that serve as guidelines for the analysis of other heritage buildings. The structural engineering group of the University of Chieti Pescara worked on the seismic safety assessment of different museums, among which was the Archaeological Museum, "*Massimo Pallottino*", located in the Norman Castle of Melfi (PZ, Italy).

The Castle of Melfi was studied in an earlier research [9], which analyzed the efficiency of the structural retrofitting by Francesco Canevaro after the 1694 earthquake. This study performs limit analyses of the historical interventions at a local level showing that the portion of the castle retrofitted by Canevaro is characterized by good seismic behavior.

In the current study, the vulnerability analysis of the museum "*Massimo Pallottino*" is performed both at a local and global level by using a linear kinematic analysis and non-linear static (pushover) analysis, respectively, according to the recommendations of the Italian "*Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage*" [6]. A more refined and complete approach (i.e., nonlinear dynamic analysis) would have required a detailed model and, consequently, deeper knowledge of the mechanical properties of materials and of construction details. In addition, the numerical modeling of masonry walls used in this study follows a macro-modeling (or continuum) approach, thus assuming that the complete masonry wall is homogeneous. A micro-modeling (discontinuous or discrete) approach is computationally expensive for large masonry structures and requires very thorough knowledge of the masonry mechanical properties [10–12]. The approach followed in the present paper, despite some inevitable approximations, still provides an efficient way to verify the safety of the masonry structure to extensive damage and collapse. All retrofitting interventions performed on the structure until now must be investigated in detail. The analysis of past interventions helps the structural analysis as it sheds light on the structural characteristics of all structural elements, without performing destructive testing on the building. Moreover, a safety assessment performed according to these principles supports the design of any strengthening intervention of the architectural heritage in order to enhance the structural safety of a structure that is structurally dated and hosts a large number of visitors, whose safety must be guaranteed.

2. Building Knowledge

According to [6], the seismic safety evaluation of heritage buildings can be performed defining a confidence factor CF as a function of the knowledge level achieved on the building. This factor is computed through the following equation:

$$CF = 1 + \sum_{k=1}^{4} CF_k \qquad (1)$$

where the four different partial confidence factors CF_k (k = 1,4) are evaluated as a function of the knowledge level for each of the following aspects: geometric survey, material survey and constructional details, mechanical properties and soil and foundations. A more accurate knowledge acquisition phase generates a lower CF to use in the structural analysis as the mean values of the masonry mechanical properties are divided by the CF and by the material partial safety factor. The structural analysis of existing buildings is indeed deeply connected to the knowledge of the building. Moreover, it encourages the designer to reach in-depth knowledge of the building in order to use higher mechanical characteristics of the structural materials. The knowledge path proposed in this work for heritage buildings is based on the same principles used for the identification of CF. In this work, they are analyzed in detail in order to define a specific and rational procedure characterized by five different steps: (1) critical-historical analysis, (2) photographic documentation and geometrical survey, (3) material survey (in situ diagnosis and laboratory tests), (4) foundation and soil survey and (5) cracking pattern and structural integrity analyses.

Figure 1 shows the general scheme of the procedure necessary to perform a complete structural behavior analysis of a historical structure. It shows that, as indicated by [13–17], the structural analysis of these structures should follow a multidisciplinary approach involving a wide variety of activities and the collaboration of different professionals such as architects, civil engineers, restorers, historians, archaeologists and building managers, among others. The structural behavior analysis (Step 2) can be performed only when all the phases required to attain good building knowledge (Step 1) are completed. For historical buildings, the time required to complete the different steps and sub-steps may vary significantly based not only on the skills of the professionals involved in this complex process (architects, engineers, restorers, test labs, etc.), but also on the time needed to find the historical documentation related to the building. Finding this historical documentation can be very time-consuming. In the case of the museum "*Massimo Pallottino*", some historical documents were provided directly by MiBACT, while others were found at the competent local offices.

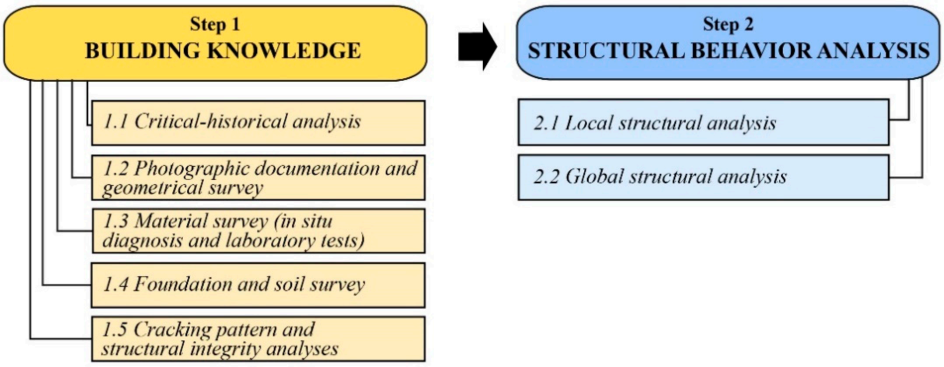

Figure 1. General scheme of the procedure necessary for a complete structural behavior analysis of a historical structure.

In the following paragraphs, each of the points needed to perform a complete structural behavior analysis is analyzed in detail for the case study of the medieval Castle of Melfi (PZ). The Castle of Melfi is one of the most important medieval castles in Southern Italy. Its origin dates back to the late 11th century and is attributed to the arrival of the Normans in Italy. The building knowledge phase is extended to the whole castle, while the structural behavior analysis is performed only for the central portion that today hosts the "*M. Pallottino*" museum. Figure 2 shows the site plan of the castle with the location of the museum and different three-dimensional views.

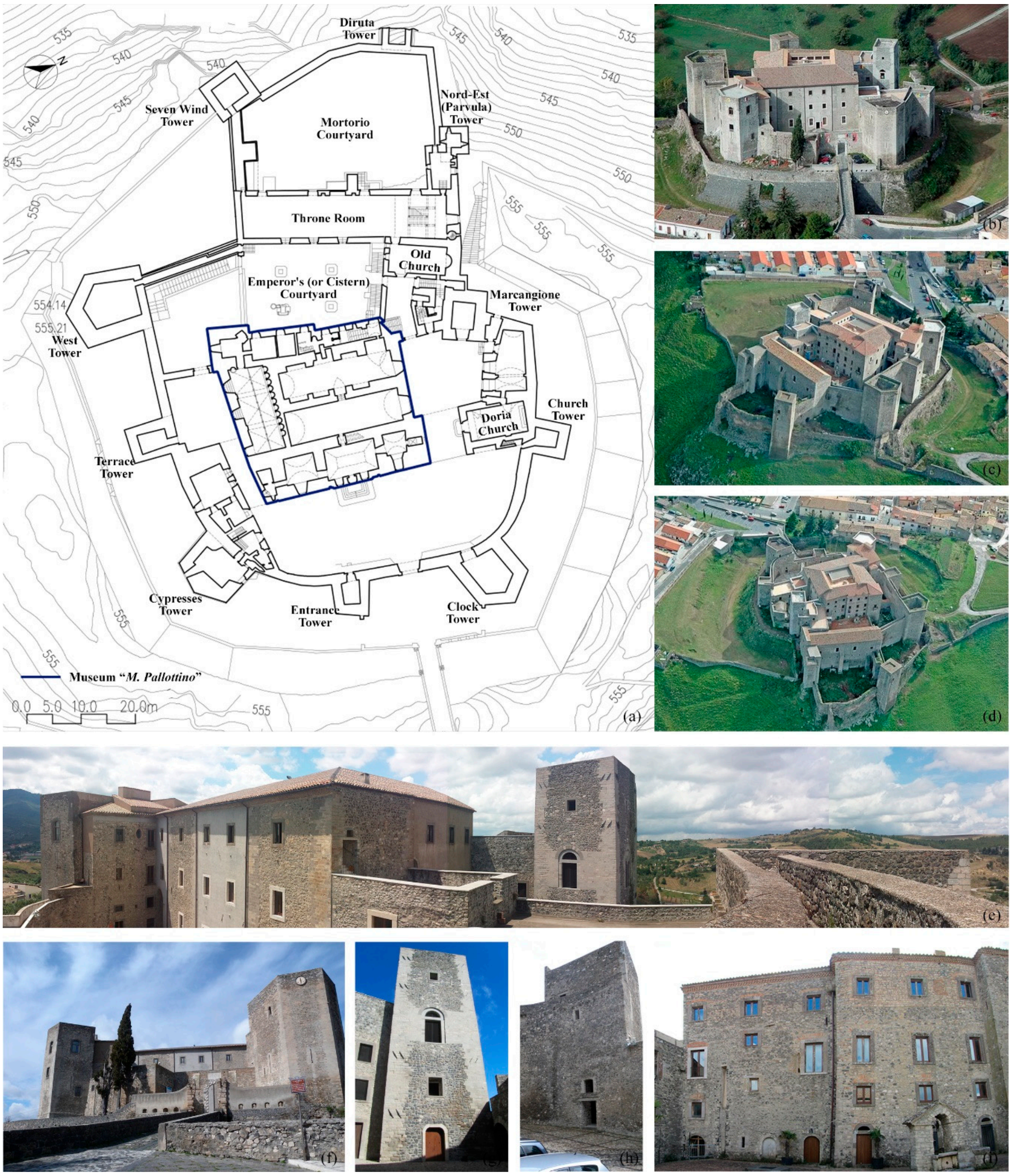

Figure 2. Site plan of the Castle of Melfi with the location of the "*M. Pallottino*" museum (**a**), three-dimensional views (from **b**–**e**), front view (**f**), Marcangione Tower and Clock Tower (**g** and **h**) and back view (**i**).

2.1. Critical-Historical Analysis

Historical information about a building is essential to understand the architectural and structural building evolution, the structural assembly of its different parts and the material differences. The history of the castle recounted by [18] reveals four constructive phases corresponding to the construction of the following castle portions (Figure 3):

Norman Building (1043–1199): Central core of the castle with three existing towers and the fourth no longer identifiable (it may have been destroyed or collapsed as a result of the continuous invasions and sieges during the Norman period).

Swabian Building (1199–1266): (a) Marcangione Tower attached to the old Norman core and the "Parvula" Tower (or Nord-Est Tower) through a long corridor for the patrol route; (b) Emperor Tower (or Seven Winds Tower) connected to the Armigeri room by narrow passages; (c) Armigeri Room which linked the Nord-Est Tower to the Emperor Tower.

Angevin Building (1266–1333): (a) Increase in height of the Nord-Est and Marcangione Towers; (b) connection between the Nord-Est and the Emperor Towers through the construction of the "Diruta" Tower; (c) increase in height of the Emperor Tower and the Armigeri Room; (d) construction of six towers (three pentagonal and three rectangular): West Tower, Terrace Tower, Clock Tower, Tower of Cypresses, Entrance Tower and Church Tower; (e) construction of a room dedicated to the collection of water (cistern) and creation of a wall connecting this cistern to the West Tower.

Doria Building (1531–1954): (a) Entrance Portal to the Castle; (b) last span of the masonry bridge of the castle entrance; (c) Dorian Chapel with an annexed guardhouse; (d) construction of internal and external rooms located against the castle walls between the Tower of Cypresses and the Terrace Tower.

The portion of the castle that currently hosts the museum (Figures 2 and 3) has mainly Norman origins, except from the entrance and two bodies attached to the north and west elevations, which probably date back to the 16th century. The above information is essential from a structural point of view, as it suggests checking the connections between the masonry walls built in different periods.

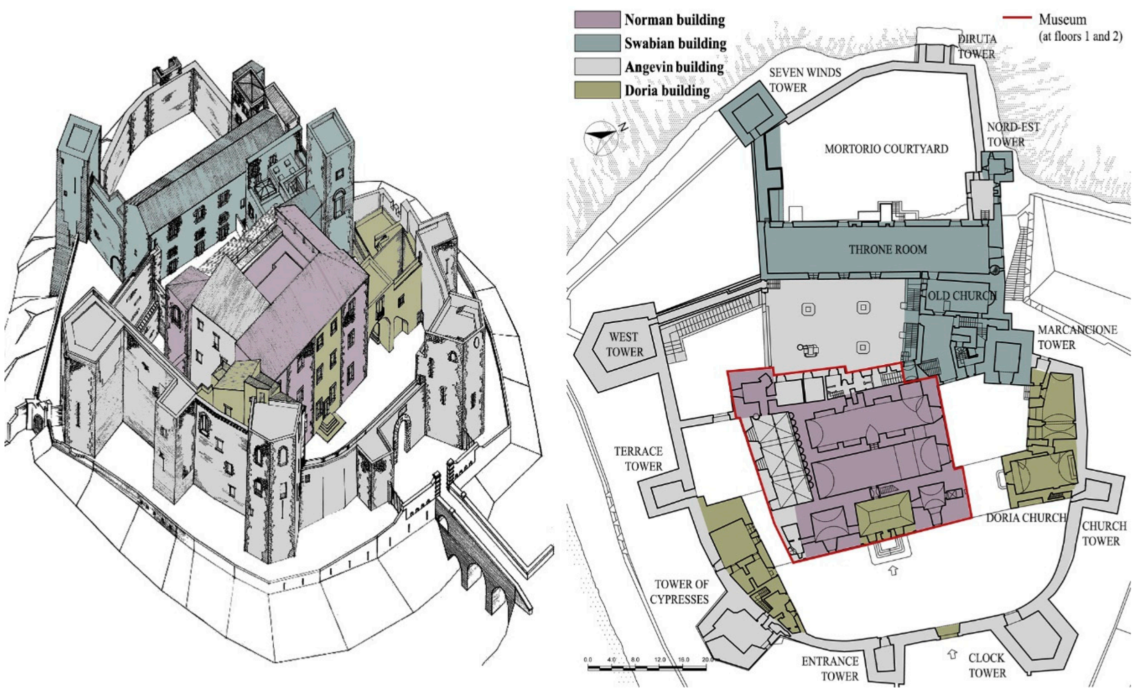

Figure 3. Constructive phases of the Castle of Melfi—axonometric view adapted from [9].

2.2. Photographic Documentation and Geometric Survey

The photographic survey can be used in each step of the building analysis and can often show details neglected during the first on-site visit. All images should be numbered, filed with relevant notes and include the photographer's position and the picture direction. The geometric survey should graphically describe the principal geometric characteristics of the construction.

Generally, survey drawings include plans, sections and elevations with the corresponding measurements. In order to perform a detailed geometric survey of the structure under study, an accurate inspection of the entire castle was performed, and a meticulous archive and documentation search was carried out in the competent public offices. This investigation revealed different geometrical surveys performed within the consolidation projects that involved the Castle of Melfi up to the beginnings of the 1990s. As an example, Figures 4 and 5 show geometric surveys dating back to the end of the 1980s within the project of post-earthquake consolidation designed and directed by Giuseppe Zampino. More specifically, Figure 4 shows the perspective view of the Nord-Est Tower and two internal sections of the Church Tower, while Figure 5 shows the perspective view of the West Tower and its geometric survey.

The comparison between the numerous surveys drafted during past consolidation interventions and the on-site surveys performed during this work enabled the authors to trace a complete geometric survey of the entire Castle of Melfi (Figure 6).

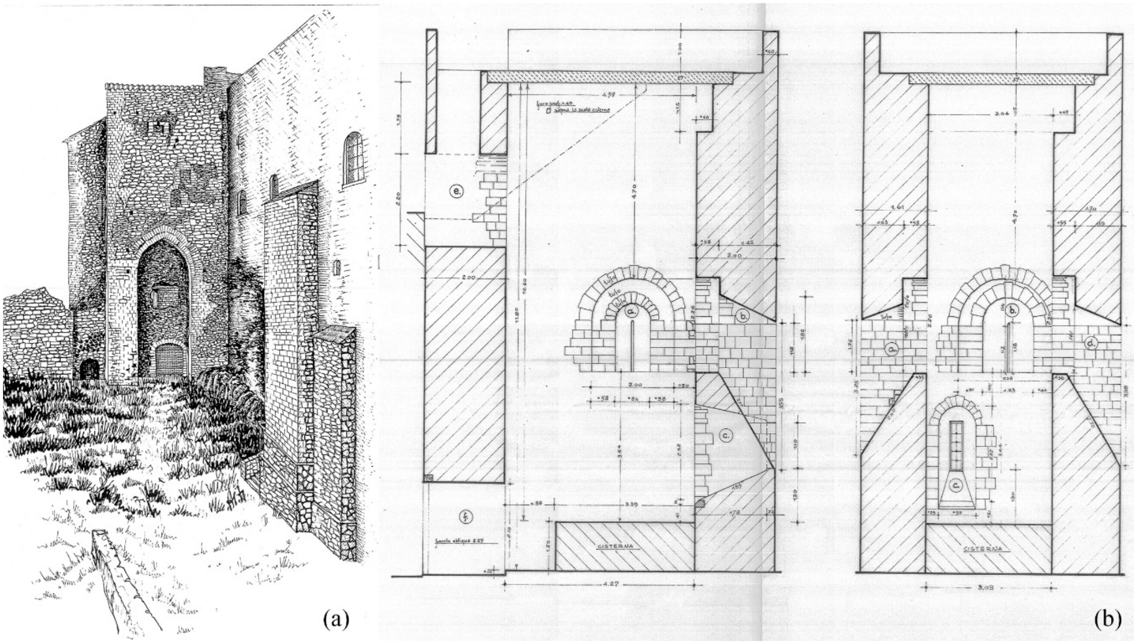

Figure 4. Perspective view of the North-East Tower (**a**) and two internal sections of the Church Tower (**b**), elaborated at the end of the 1980s within a project of post-earthquake consolidation.

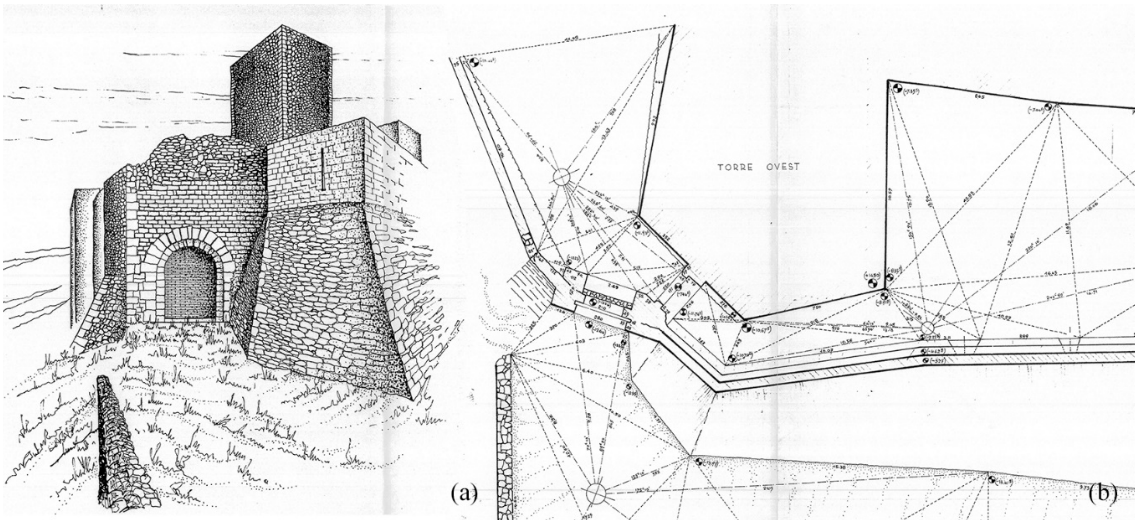

Figure 5. Perspective view (**a**) and geometrical survey (**b**) of the West Tower, elaborated at the end of the 1980s within a project of post-earthquake consolidation.

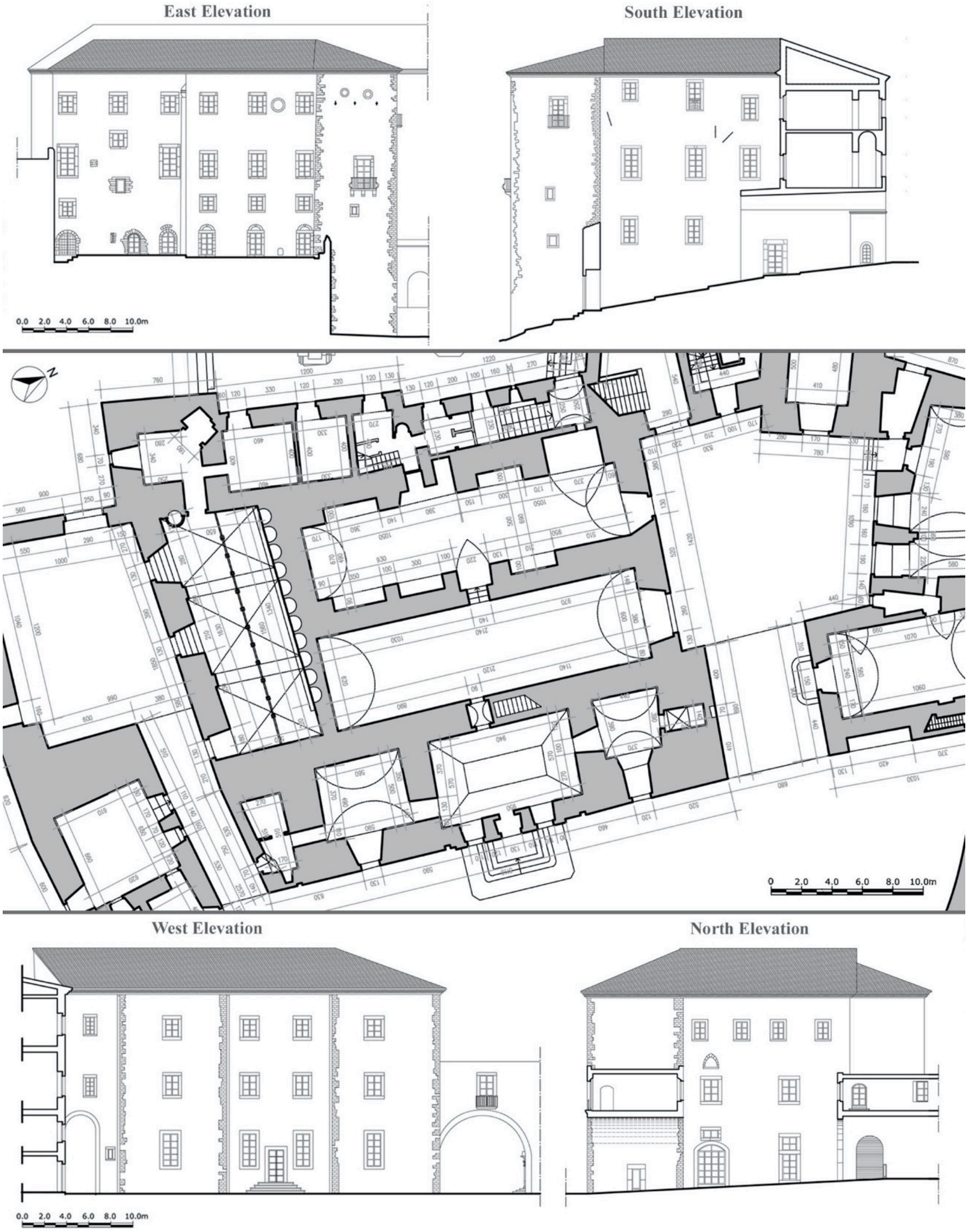

Figure 6. Geometric survey of the ground floor and the elevations of the central part of the Castle of Melfi (PZ), Italy.

2.3. Structural Identification and Material Survey

A heritage building often consists of different parts erected in different historical periods and characterized by different structural characteristics and material properties. This phase is carried out after or alongside an accurate historical-critical analysis, during which all information on the construction and transformations undergone by the building is collected. Furthermore, the building may have been subjected to consolidation interventions in recent times, but these interventions may not be documented and are not found during the critical-historical analysis. These interventions can deeply modify the original material characteristics and constructive systems; thus, their identification is indispensable to reach adequate knowledge of the building.

The historical-critical analysis of the Castle of Melfi showed four different construction ages of the castle but did not provide any information on the recent consolidation interventions. Further, earthquakes in 1694, 1731, 1805, 1857, 1930 and 1980 caused widespread damage to the city and the castle. Table 1 reports all earthquakes with an intensity greater or equal to seven on the Mercalli–Cancani–Sieberg macroseismic MCS scale that affected the site. The data on the event and the intensity I at the site are derived from the Italian Parametric Earthquake Catalogue CPTI15-DBMI15 [19], while the epicentral distance to the castle is calculated in relation to the specific coordinates of the castle (Latitude 40.998; Longitude 15.653).

Table 1. List of the earthquakes felt in Melfi with intensity I ≥ 7. The seismic events are extracted from [19]: I = intensity in Melfi (scale MCS); I_0 = epicentral intensity; M_w = moment magnitude.

I	Event Coordinate		Epicentral Distance (km)	Year	Month	Day	Epicentral Area	I_0	M_w
	Lat.	Long.							
8	41.302	14.711	85.79	1456	12	05	Appennino centro-merid.	11	7.19
8	40.862	15.406	25.68	1694	09	08	Irpinia-Basilicata	10	6.73
7	41.274	15.757	31.88	1731	03	20	Tavoliere delle Puglie	9	6.33
10	40.96	15.669	4.47	1851	08	14	Vulture	10	6.52
8–9	40.994	15.653	0.48	1851	08	14	Vulture	7–8	5.48
7	40.352	15.842	73.61	1857	12	16	Basilicata	11	7.12
7	40.898	15.421	22.44	1910	06	07	Irpinia-Basilicata	8	5.76
9	41.068	15.318	29.14	1930	07	23	Irpinia	10	6.67

Ref. [20] reported that the castle was damaged by the 1456 earthquake, too. More detailed information on damage caused by earthquakes is available starting from the 1694 Irpinia earthquake. Ref [9,21] reported that the castle was severely damaged by this earthquake. The Doria family called Genoese Francesco Canevaro to repair the castle and reinforce the building. F. Canevaro introduced pillars and walls and added tie-bars, mainly in the central core of the castle. The tie rods are intended to equilibrate the thrust forces (for example, the vaults' unequilibrated forces) that could otherwise destabilize, out of plane, the external walls or columns. The double earthquake of 1851 induced the collapse of the Seven Winds Tower and caused severe damages to the Marcangione Tower and the Throne Hall [20,22]. These collapses were followed by demolitions and reconstructions, including some of the walls of the Throne Hall. The 1930 earthquake caused significant damage to the Castle (Figure 7), more specifically in the vaults, in the Seven Winds, Cypresses, Clock and Church Towers, in the façade on the cistern courtyard and on the east end of the main façade [18]. Following the quake, the Doria family commissioned Architect L. Lenzi to restore the castle to its original architectural splendor.

Figure 7. The Castle of Melfi after the earthquake of 23 July 1930.

The restoration works reported by [18] are the following: (a) liquid cement injections into the masonry cracks; (b) demolition and reconstruction of the unstable and unsafe walls with stone and cement-based mortar, including courses of bricks; (c) introduction of longitudinal and transversal tie-rods in the central part of the castle; (d) substitution of the damaged vaults with horizontal slabs consisting of iron girders and hollow clay floor slab blocks; (e) construction of reinforced concrete ring beams under part of the roof; (f) demolition and substitution of the stairs at the second floor with a large ramp and (g) construction of stone parapets on the bridge and the courtyards. During these works, L. Lenzi found in the archives of the Doria family the oldest drawings of the castle by the topographer F. Canevaro (Figure 8) dating back to 1695. These drawings are still today a fundamental instrument to understand the castle structural configuration in the second half of the 17th century.

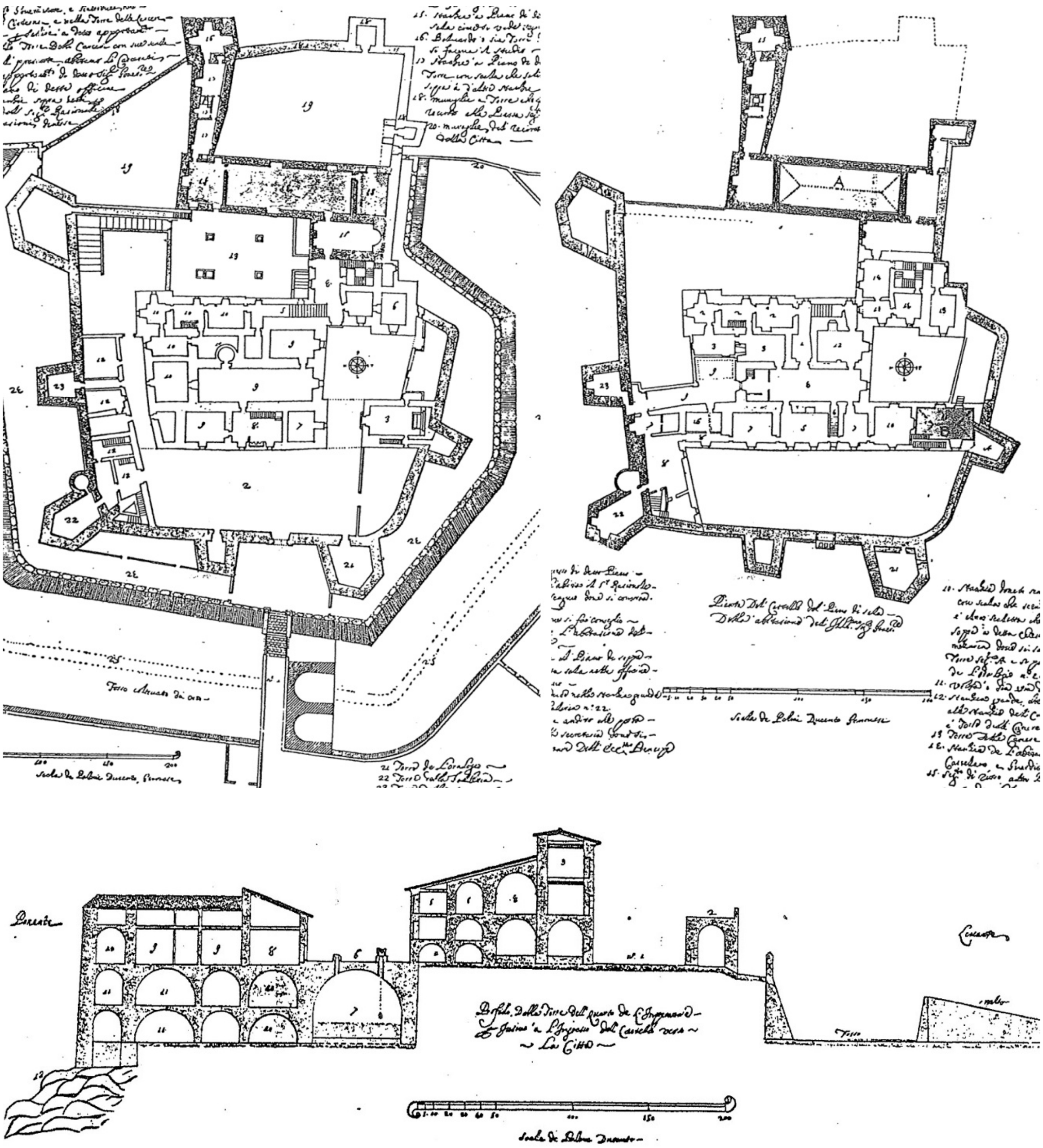

Figure 8. Ground floor plan, first floor plan and longitudinal section of the Castle of Melfi drawn in 1695 by F. Canevaro (from the archives of the Doria family).

The first important structural intervention was carried out after the donation of Prince Andrea Doria Pamphilj to the Italian State in 1952 took place between 1965 and 1968. The walls surrounding the castle, the Armigeri Room, the overhead Throne Room and the older central part of the castle with Norman origin were restored. Historical sources reported an accurate description of the work carried out in these years for the Throne Room. More specifically, in 1968, the barrel vault that supported the floor level was completely demolished. A new two-pitched roof was built, and the original nineteenth-century transverse wall was demolished and rebuilt following the walls at the lower level. Moreover, the jambs of several large windows were discovered under the plaster.

In 1970, the castle became host to a museum. This change required the structural consolidation and restoration of the castle's central core. Retrofitting on this portion of the castle was performed from the late 1970s to the early 1990s and included: (1) a new roof with double steel girders, corrugated sheets and reinforced concrete slabs and ring beams connected to the masonry walls, (2) grout injections of the masonry walls, (3) strengthening of the vaults, (4) masonry corner reinforcement with steel bars and (5) replacement of old floors with steel girders and reinforced concrete slabs. They were installed independently of the old wooden beams which, after appropriate restoration, remained in place as support for the lower wooden ceiling. Figure 9 shows a few construction details included in the original 1988 drawings.

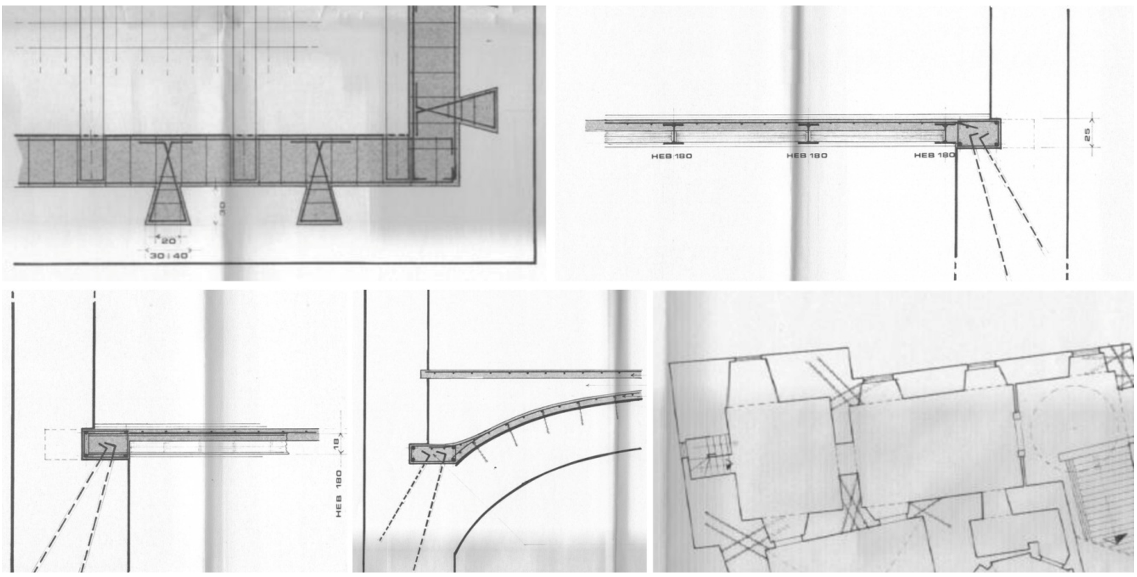

Figure 9. Some construction details included in the project of restoration of the central part of the castle (from design documentation: arch. Giuseppe Zampino, 1988).

During the above retrofitting, material tests were planned in different parts of the castle. The most relevant campaigns were performed in 1990 and in 1999. In the first one, Professors P. Rocchi and G. Morabito carried out deformation measures of cracks, in situ tests with single and double flat jacks and laboratory compression tests on masonry specimens. The second campaign by Professor P. Rocchi focused not only on the masonry walls, but also on the floors and vaults. The following tests were performed: 2 flat jack tests, 5 grout injection tests, 2 floor load tests to evaluate the vault behavior, 13 exploratory tests on masonry walls and floors and 19 endoscopic analyses performed on the masonry walls, vaults and floor slabs at each level of the structure. Figure 10 shows the location of each test performed during the 1999 campaign and the results of four exploratory tests. Professor Rocchi's report also contains photographic documentation of the castle and the images of several tests (Figure 10 top left and Figure 11). Results of these tests are used in this work to evaluate the construction characteristics of the castle, the permanent loads and the material mechanical properties.

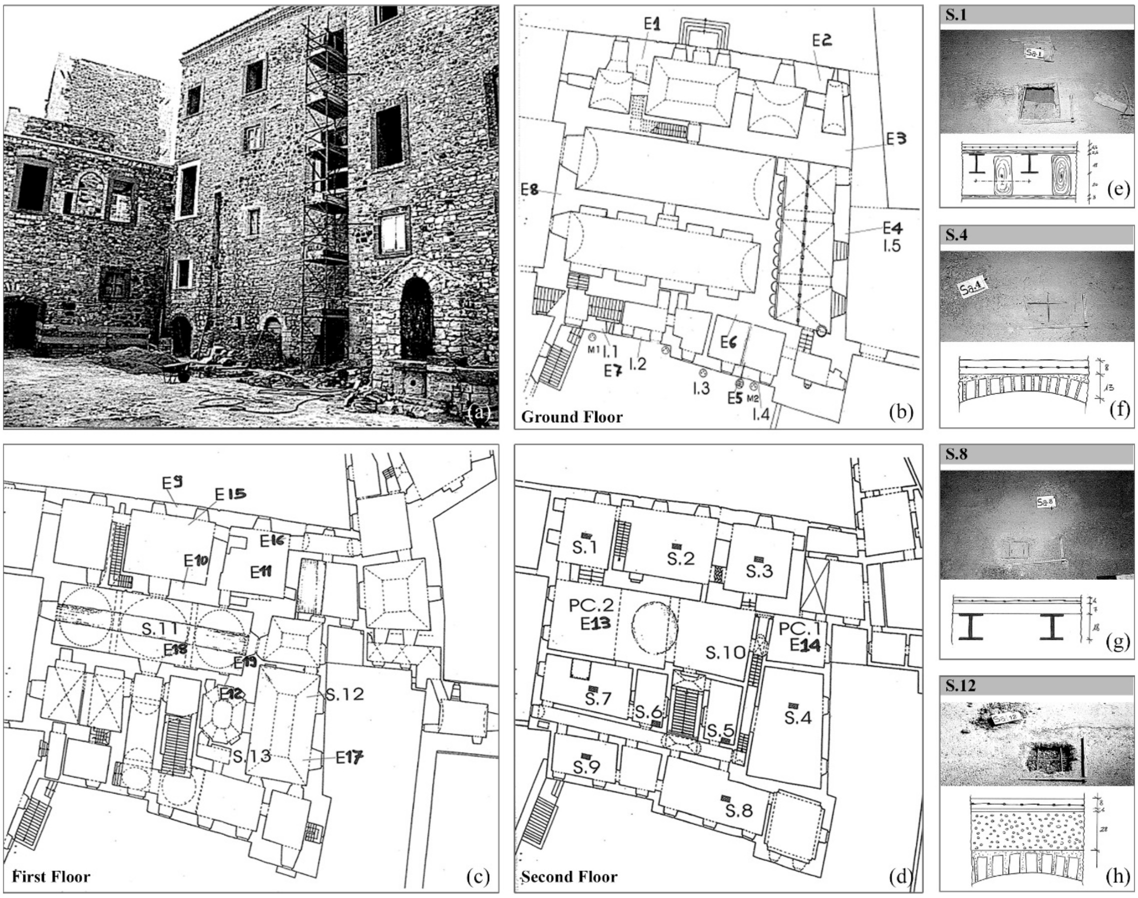

Figure 10. The castle during the 1999 survey (**a**), location of each test performed at the ground floor (**b**), the first floor (**c**) and the second floor (**d**) and results from 4 exploratory tests (from **e**–**h**). The symbols used for the location of the tests are: M = flat jack tests, I = grout injection tests, PC = floor load tests, S = exploratory tests and E = endoscopic analyses (images adapted from the results of the 1999 survey by the Professor P. Rocchi).

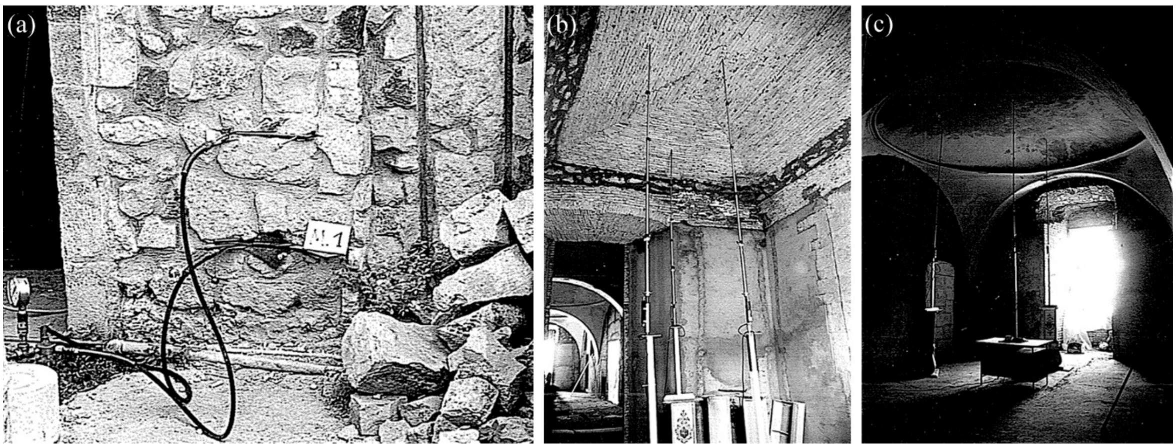

Figure 11. Photographic documentation of the double flat jack tests (**a**) and the floor load tests (**b** and **c**) from the 1999 survey by Professor P. Rocchi.

The masonry mechanical properties of the central part of the castle, which currently hosts the "*M. Pallottino*" museum, were first defined according to the recommendations of the Italian building code [23,24] and were then compared with the results of the experimental tests. The safety evaluation of the central part of the castle is performed using a mean compression strength f_m = 1.40 N/mm^2, mean shear strength equal to 0.026 N/mm^2, elastic modulus of 870 N/mm^2 and shear modulus of 290 N/mm^2. According to [6], masonry mean strength is divided by a knowledge factor CF = 1.20 and a partial safety factor γ_m = 2.

2.4. Foundation and Soil Characterization

The geological and geotechnical characterization of the soil of the Castle of Melfi is based on the soil surveys carried out during the seismic retrofitting by Professors G. Morabito and P. Rocchi in the early 1990s. The castle foundations rest on soil characterized by a friction angle ϕ = 25°–30° and a cohesion C = 200–300 kPa. According to the European seismic code [25], this is Ground Type B: *Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth*. Furthermore, a topographic amplification factor S_T = 1.2—corresponding to "*Isolated cliffs and slopes*"—was used.

The foundation of the castle is characterized by masonry walls that extend below the ground level. The surveys carried out to date have not given further information on their depth and width.

2.5. Cracking Pattern and Structural Integrity Analyses

The visual inspection of the Castle of Melfi showed that the central part of the castle is in a good conservation state, thanks mainly to the restoration campaigns of the last few decades. However, the detailed investigation of the ancient and recent castle history and its historical photos (Figure 12) show that in the second half of the 1970s, the castle was in a very poor state of maintenance. When historical buildings do not undergo regular maintenance, environmental actions may deteriorate all structural components, including the roof and the walls, as indicated in [26]. It is imperative to identify the actual sources of decay and vulnerability (such as lack of maintenance, prolonged roof leakage, poor initial design, excessive loading, foundation settlement . . .) in order to plan appropriate interventions, that may be structural, such as wall reinforcement, or non-structural, such as roof fixing.

Figure 12. Historical photos of the Castle of Melfi (PZ) in the second half of the 1970s (from MiBACT Archives).

The distinction between different decay phenomena dates back to Alberti [27], which identifies the causes of the different degradation processes. In this work, the decay survey was performed according to [28], which proposes a specific decay glossary with the associated graphic symbol for each phenomenon. The graphic decay survey was integrated with the description of the decay phenomena, the analysis of their principal causes and their photographic reproduction. Figure 13 shows the decay and cracks survey carried out on the central core of the castle. This survey showed that there are no major cracks and that widespread surface deposits are present, along with different discolorations and limited traces of moisture, washout, vegetation, biological colonization, alveolarization and disintegration.

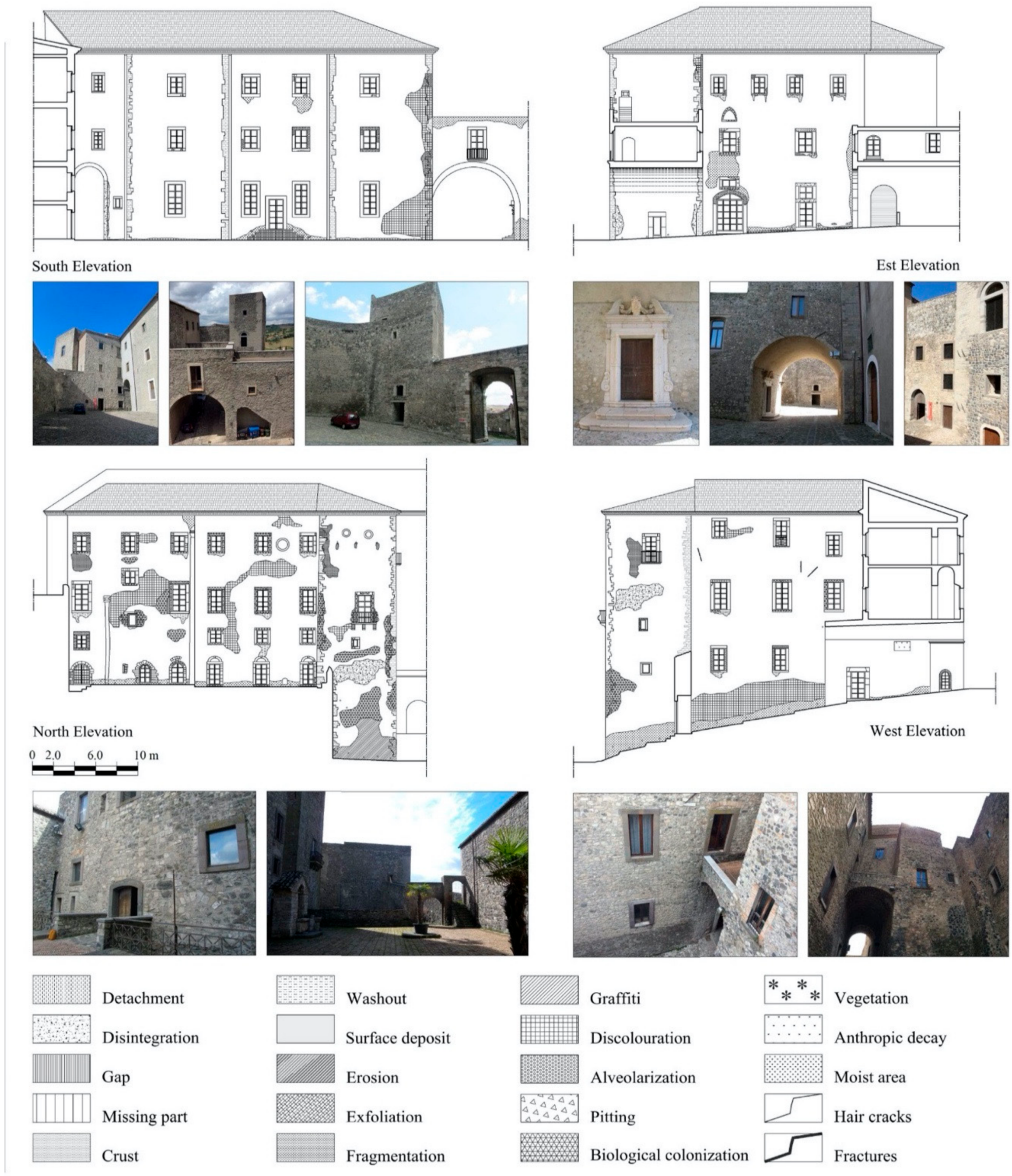

Figure 13. Decay and cracks survey of the east elevation and the part of the north elevation with the greatest signs of decay.

3. Structural Behavior Analysis

Masonry walls can fail following both in-plane and out-of-plane failure mechanisms. According to [29], the "First Damage Mode" is produced by seismic actions perpendicular to the wall (out-of-plane). Direct observation of post-earthquake crack patterns showed that these mechanisms represent the highest building vulnerability [30]. Only if the walls are connected with each other and with the horizontal floors can the "Second Damage Modes", corresponding to an in-plane failure mechanism, be activated. The first damage modes of the museum "*M. Pallottino*" were analyzed through a kinematic linear analysis on the walls most vulnerable to the activation of local mechanisms of collapse. More specifically, the seven most vulnerable walls are identified on the external perimeter of the museum (Figure 14, left). For each analyzed wall, different local failure mechanisms were considered depending on the localization, the thickness and the strengthening interventions of the wall (Figure 14, right). As an example, Wall 1 was checked for vertical out-of-plane bending. This local mechanism is activated when a slender wall is restrained at the ends only and is free in the central span. The end restraints are typically effective in preventing global overturning of the wall, but a crack at the mid-height of a slender wall may occur, inducing out-of-plane instability. In the case of Wall 1, even though a ring beam was inserted at each floor level during the 1970s works, the out-of-plane instability of the outer wall leaf is not prevented as the ring beams are not effectively connected to the outer leaf.

The knowledge phase showed that the structure is currently equipped with ring beams connected to the masonry walls with steel bars having a length approximately equal to 1.20 m. This condition inhibits the activation of the overturning mechanisms, even if the vertical out-of-plane bending mechanisms are not excluded. Moreover, as the museum's walls are characterized by double-leaf stone masonry, where the wall is the thickness (e.g., at the lower levels), the steel bars might not extend until the external wall leaf, and in this case they would be unable to prevent the activation of the vertical out-of-plane bending of the external wall leaf for two or more levels (Figure 14, right).

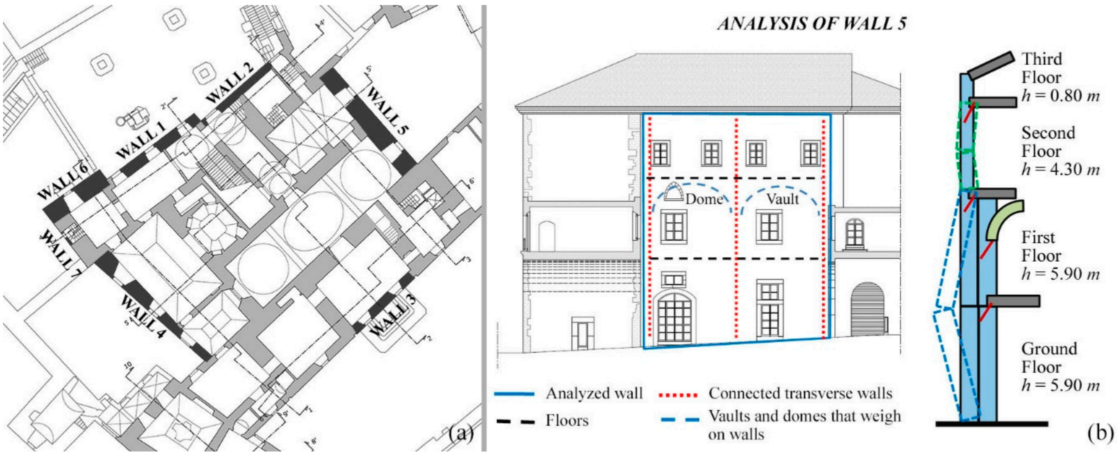

Figure 14. Identification of the walls analyzed by the kinematic linear analysis (**a**) and scheme for the identification of the forces and the structural mechanisms involved in the kinematic analysis of Wall 5 (**b**).

Table 2 shows the results of the kinematic analyses. For each considered wall, the following parameters are shown: the analyzed mechanism type with the floors involved (vertical out-of-plane bending of the whole wall or of the single external wall leaf), the peak ground acceleration (PGA) associated with the activation of the considered kinematic mechanism ($a_{g,\mathrm{ULS}}$) at the ultimate limit state (ULS), the seismic demand characterized by the target peak ground acceleration ($\mathrm{PGA}_{\mathrm{ULS}}$ on rigid soil) and the capacity/demand ratio. The seismic demand is computed (i.e., the expected maximum horizontal acceleration) considering the site geographic coordinates and its stratigraphic and topographic characterization. The strategic relevance of the building and its expected use is included using a design ground motion return period at the ultimate limit state T_R = 712 years. For the given site and return period, the peak ground acceleration on rigid soil a_g is 0.232 g.

Table 2. Results obtained from the analysis of the out-of- plane failure mechanisms.

Wall. *n.*	Mechanisms Type	Capacity	Demand	Ratio
		$a_{g,\mathrm{ULS}}$(g)	$\mathrm{PGA_{ULS}}$ (g)	α_{ULS}
1	Vertical out-of-plane bending of the external wall leaf (Ground Floor, Mezzanine and First Floor)	0.474	0.232	2.043
	Vertical out-of-plane bending of the external wall leaf (Ground Floor, Mezzanine, First and Second Floor)	0.350	0.232	1.509
	Vertical out-of-plane bending of the wall (Second Floor)	0.488	0.232	2.103
2	Vertical out-of-plane bending of the external wall leaf (Ground Floor and First Floor)	0.379	0.232	1.634
3	Vertical out-of-plane bending of the wall (Ground Floor)	2.226	0.232	>>1
	Vertical out-of-plane bending of the external wall leaf (Ground Floor)	2.229	0.232	>>1
	Vertical out-of-plane bending of the external wall leaf (Second Floor)	0.615	0.232	2.651
	in	0.181	0.232	0.780
4	Vertical out-of-plane bending of the external wall leaf (Ground Floor)	0.437	0.232	1.884
	Vertical out-of-plane bending of the wall (Second Floor)	0.739	0.232	3.185
5	Vertical out-of-plane bending of the external wall leaf (Ground Floor and First Floor)	0.335	0.232	1.44
	Vertical out-of-plane bending of the wall (Second Floor)	0.660	0.232	2.845
6	Vertical out-of-plane bending of the external wall leaf (Ground Floor, Mezzanine and First Floor)	0.584	0.232	2.517
7	Vertical out-of-plane bending of the wall (Stairwell floor)	0.144	0.232	0.621

Due to the complexity and the extension of the building under study, the analysis of the global structural behavior was evaluated using the simplified "equivalent frame" modeling approach. The structural model is made of beam elements with rigid links at the intersection between the masonry piers and the horizontal spandrels [31–33]. Several studies [34,35] have investigated and verified the suitability of linear and nonlinear equivalent frames for modeling unreinforced masonry structures.

In this work, the equivalent frame model and the global analyses were performed with the Aedes PCM software [36]. The analytical model of the castle takes into account the experimental mechanical properties founded in the material survey, according to a hybrid vulnerability assessment method [37,38]. The geometric model comprises only the portion of the castle that currently hosts the museum (Figure 15). The structural model considers the floors as rigid diaphragms. The same modeling strategy and the same software were used for the seismic vulnerability assessment of other relevant monumental buildings, such as the Civic Museum of Sansepolcro (Italy) that contains an important fresco painted by Piero della Francesca [39], the Albornoz fortress, a 14th stone masonry construction located in central Italy [40], the Palace of Priors in Perugia (Italy), a medieval monumental building that currently hosts an important museum [41] and the Pompeii's Stabian Baths where the model was used to assess the mechanical behavior of selected masonry structures damaged during 1st century seismic events [42].

Since the central core of the castle is not an isolated structure, the constraining action of the adjacent structures was simulated restraining the horizontal translations of the nodes connected to the adjacent building. This approximation is, however, limited to very few points, as the points of contact with adjacent structures are few.

In the present work, vaults and domes are not explicitly included in the global model: only their weight is considered. In order to obtain a detailed evaluation of these loads, single vaults and domes were modeled in a separate finite element model by using two-dimensional finite element models with the MIDAS Gen [43] software (Figure 16). The approach is on the homogenization of the material properties of blocks and mortar [44,45]. The main scope of this detailed analysis was to determine how the vaults and domes transmit the loads to the supporting vertical elements.

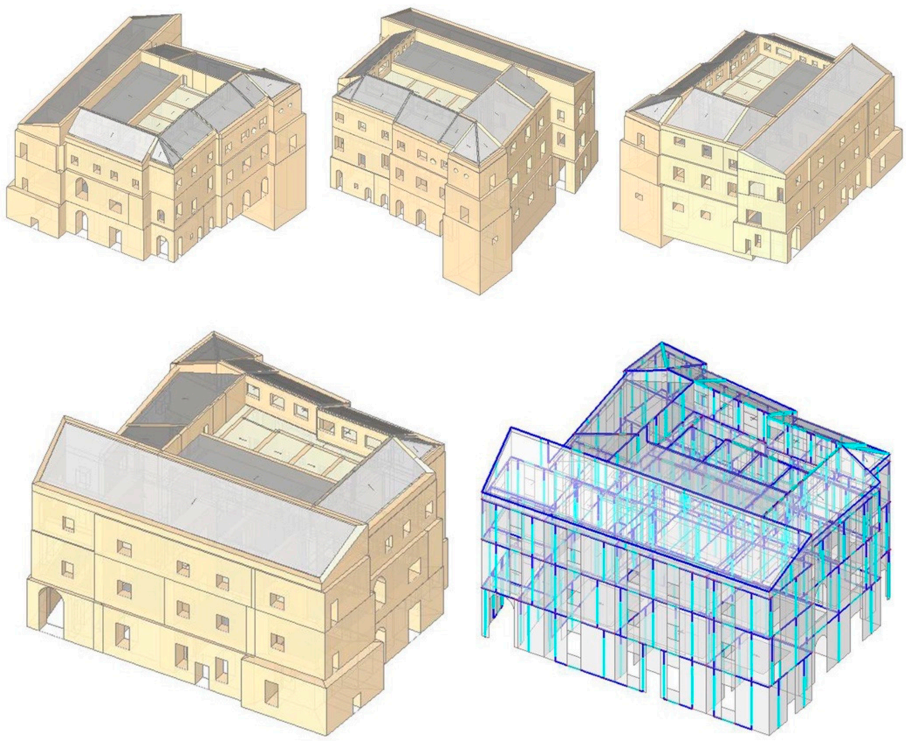

Figure 15. Geometrical model implemented with the Aedes PCM 2014 software and structural model performed using the "equivalent frame" modeling approach.

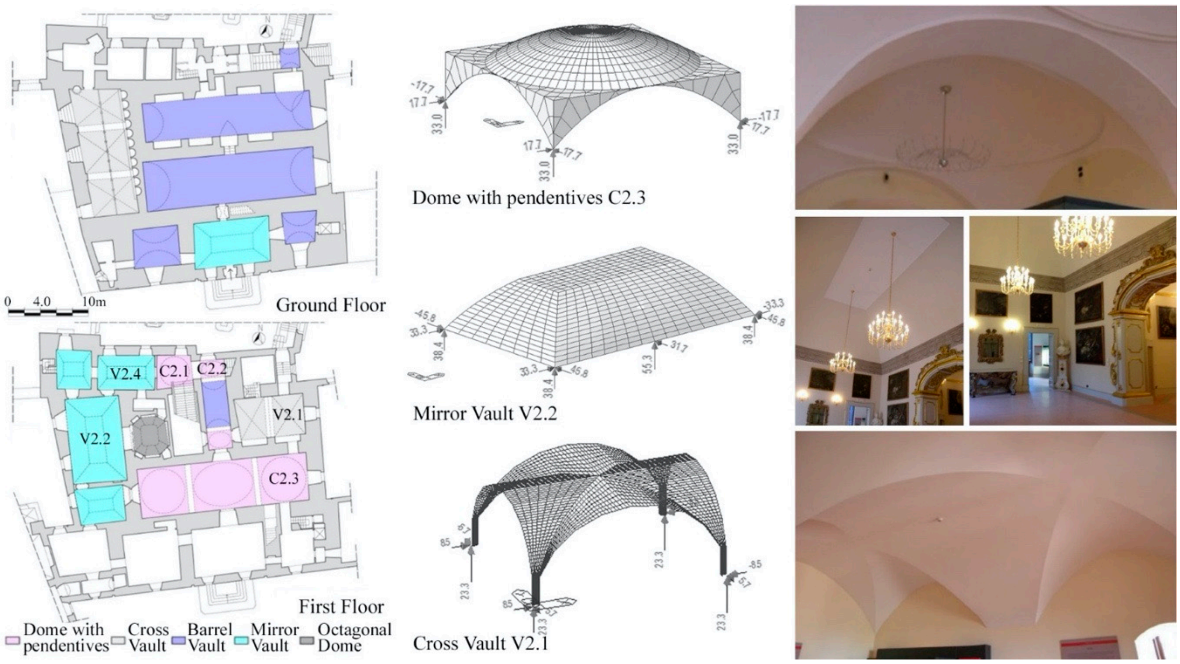

Figure 16. Two-dimensional finite element model of a vault typology performed with the software MIDAS Gen [43].

The modal analysis of the global model shows that the first three vibration modes of the structure are equal to T_1 = 0.273 sec, T_2 = 0.255 sec and T_3 = 0.219 sec, respectively (Figure 17). The first two modes are mainly translational, while the third mode is mainly torsional.

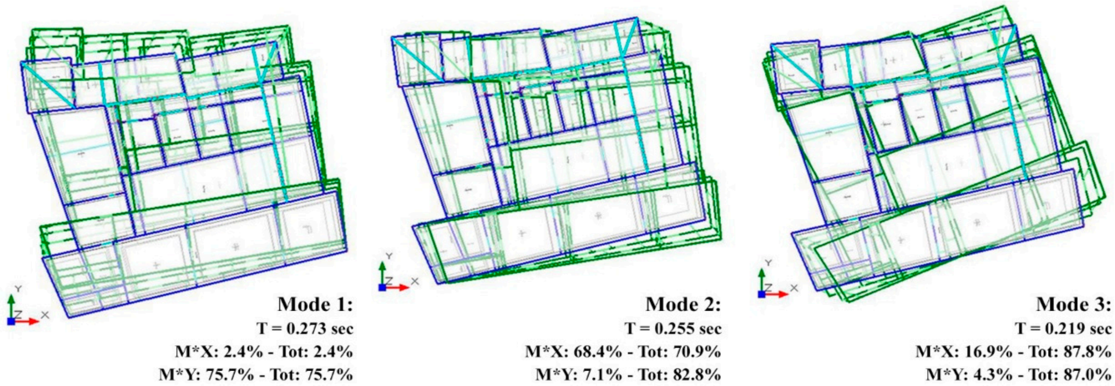

Figure 17. The first three vibration modes of the case study structure.

The global analysis of the structure was performed with a nonlinear static (pushover) analysis [46]. The seismic demand was evaluated according to the N2 method as originally proposed by Fajfar [47] and found in Eurocode 8 [25]. The bending and shear nonlinear behavior of piers and spandrels were modeled by plastic hinges located at both ends of the elements and characterized by elasto-plastic laws with ultimate displacements equal to 0.6% and 0.4% of the element height, respectively [23]. Following Eurocode 8 [25], the pushover analyses were performed considering two force distributions, a "uniform" pattern proportional to the masses (distribution E) and a "modal" pattern proportional to the lateral force distribution determined with the linear static analysis (distribution A). Each force distribution is applied in the longitudinal (X) and transverse (Y) directions, with both positive and negative signs. For each direction and force distribution, the corresponding shear-displacement curve of the multi-degree of freedom (MDOF) system is obtained and then transformed into a bilinear elasto-perfectly plastic curve corresponding to an equivalent single degree of freedom (SDOF) system. The bilinear equivalent curve is obtained considering the building lateral capacity until a 20% reduction in the maximum base shear. Finally, the obtained curves are compared with the seismic demand characterized by the elastic spectrum of the reference site. Table 3 shows the results of the eight capacity curves corresponding to each force distribution and the relevant safety indices at the ultimate ($\alpha_{ULS,PGA}$) and damage limit states ($\alpha_{DLS,PGA}$), respectively. These indices are calculated as ratios between the PGA associated with reaching the considered limit state (seismic capacity) and the target acceleration corresponding to the reference site and structure (seismic demand).

Table 3. Safety indices obtained for the ultimate limit state ($\alpha_{ULS,PGA}$) and the damage limit state ($\alpha_{DLS,PGA}$).

Curve *n.*	Force Distribution	Direction	$\alpha_{ULS,PGA}$	$\alpha_{DLS,PGA}$
1	Triangular force distribution (A)	+X	0.375	1.000
2	Triangular force distribution (A)	−X	0.358	1.064
3	Triangular force distribution (A)	+Y	0.302	0.897
4	Triangular force distribution (A)	−Y	0.293	0.859
5	Uniform force distribution (E)	+X	0.655	1.449
6	Uniform force distribution (E)	−X	0.892	1.282
7	Uniform force distribution (E)	+Y	0.453	1.231
8	Uniform force distribution (E)	−Y	0.453	0.962

Considering all the combinations and all the seismic directions requested by the Italian and European building codes [23–25], the most severe loading condition is in the y-direction (-Y seismic action) for a triangular force distribution. Figure 18 shows the pushover curve for this load direction, the force distribution and the shear status of the masonry walls at the step corresponding to the (a) ULS and (b) the last step of the analysis. This result is determined by the presence of few bearing walls in the transversal direction (y-direction), and by the dead loads on floors that are mainly distributed to the walls parallel to the longitudinal direction (x-direction). The central part of the castle served as the Doria family's residence from 1531 to 1954. During this period, the walls' distribution of the first floor was significantly changed, introducing a significant irregularity due to the construction of walls in the Y direction that are not extend vertically from the ground level to the first floor but transfer the vertical load on a barrel vault. Though this condition was partially corrected by the consolidation works at the end of the 1980s, which involved the insertion of a double reinforced concrete beam at the base of the wall rigidly connected at the vault, the walls' irregular distribution is the main cause of the structure vulnerability.

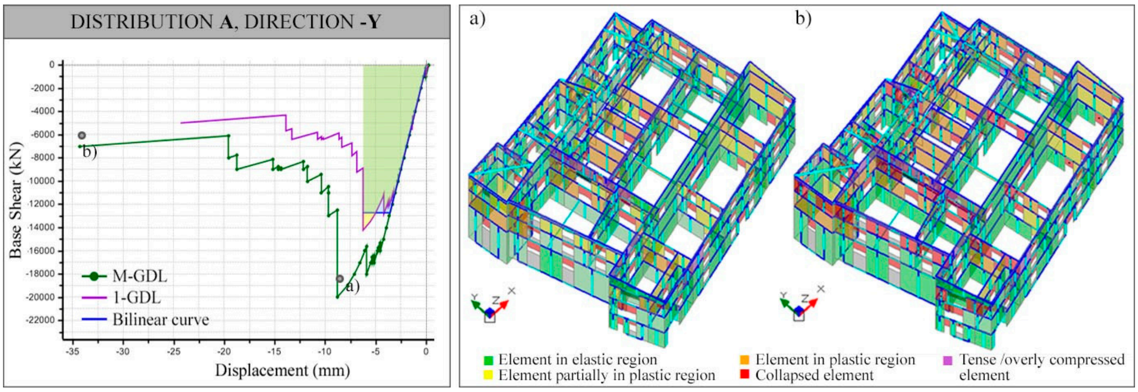

Figure 18. (**Left**): capacity curves from the pushover analysis with a triangular force distribution A negative Y direction; (**Right**): shear status of the masonry walls at the step corresponding to (**a**) the ultimate limit state (ULS) and (**b**) the last step of the analysis.

According to the 2008 Italian building code [23], another useful parameter for the verification of the structure is q^*, i.e., the ratio between the demand total base shear on the building, assumed elastic, and the yielding strength of the equivalent nonlinear system (with the limitation that $q^* < 3$). The q^* values for the different load distributions range between 2.858 and 4.815 and, coherently with the results obtained in terms of $\alpha_{ULS,PGA}$, in this case, too, the most unfavorable condition ($q^* = 4.815$) is observed in the y-direction for a triangular force distribution. It is expected that when the safety factor is less than 1, the corresponding coefficient q^* is larger than 3 [48].

4. Ideas for Structural Strengthening of the Castle

Recent seismic events (e.g., Athens 1999, L'Aquila 2009, Emilia Romagna 2012, Central Italy 2016, Lesvos 2017) have confirmed the high vulnerability of historical unreinforced masonry structures under horizontal actions, clearly indicating the need for interventions that would safeguard the lives of the users, as well as their cultural and heritage value. The retrofit of architectural heritage should be based on a combination of performance-based and conservation criteria. Retrofitting should rely on materials that are compatible with the existing ones and should comply with the principles of minimum intervention as defined by existing conservation guidelines [5].

As described in the previous paragraphs, the historical analysis of the expansions undergone over the centuries by the building that today hosts the "*M. Pallottino*" museum lead to the definitions of accurate structural models at both the local and global levels. These models consider and include most of the structural strengthening interventions performed during the last 500 years of the castle's life, including the insertion of rod-ties to prevent the walls' out-of-plane overturning, grout injections to enhance the mortar mechanical properties, insertions of floor diaphragms to ensure the building box-like behavior, vaults' strengthening and masonry corner reinforcement with steel bars.

The results of the seismic safety evaluation of the museum show that past interventions on the building guarantee good structural behavior of the masonry walls at the local level (prevention of out-of-plane mechanisms). However, some elements are still quite vulnerable. For example, the external leaf of Wall 3 (Table 2) appears to be vulnerable to out-of-plane bending. The external leaf should be connected to the other leaves and to the floor diaphragms using appropriate connectors. These connectors should be of materials compatible with the existing masonry of the castle such as steel fiber thread connectors injected with natural mortar compliant lime (see for example [49]).

Due to the addition of a staircase inside Wall 7 (see Figure 14), the wall could be subjected to vertical out-of-plane bending. In this case, the two sections of the wall currently separated by the staircase should be connected at the intermediate staircase landings using steel connectors injected with natural mortar compliant lime.

As for most heritage buildings, for earthquakes at the ULS, the global analyses show that the castle is vulnerable to in-plane flexural and shear failure of the walls. First of all, specific in situ tests are needed to confirm the presence of the consolidation interventions carried out in the 1980s, and their mechanical consistence assessed. If necessary, additional grouting could be injected. External strengthening with steel of plastic fibers is typically unpractical in this castle, because its high historical and cultural values prevent the use of new materials that would considerably alter the walls' original stone texture. Finally, in order to redistribute structural loads and masses, the storage of materials in the attic could be relocated on the lower floors. The above interventions are consistent with the conservation principles and have a minimal impact on the architectural heritage while allowing an effective structural rehabilitation.

It is easy to understand that heavy structural strengthening is not feasible for historical buildings and therefore seismic enhancement rather than full compliance with modern seismic codes safety levels is the goal of interventions on historical buildings of considerable cultural value such as Melfi Castle. Strengthening is a compromise between conservation of the original architectural and structural systems and enhancement of the building seismic safety.

5. Conclusions

The seismic safety evaluation of a heritage building should be performed using different interrelated phases that include both the building knowledge and the investigation of its structural behavior. The application of this approach to the museum "*M. Pallottino*" located in the Castle of Melfi (Italy) pointed out that the building knowledge phase is a fundamental step toward the evaluation of its structural behavior. An accurate critical-historical analysis, a detailed investigation on previous geometrical, material and soil surveys and a meticulous individuation of the crack pattern allow to minimize the structural surveys (and thus intrusive and possibly destructive tests) on the monumental building, thus preserving its historical and cultural identity. Moreover, the accurate evaluation of the building structural evolution over time limits the costs for the knowledge phase while leading to a more accurate seismic analysis of the structure.

In this work, both global and local failure mechanisms of the structure under study were analyzed. The analysis of the global failure mechanisms was performed using a simplified global model based on the "equivalent frame" approach which was deemed applicable due to the detailed understanding of the building overall structural behavior obtained following the initial detailed study of the building evolution and conservation state. The behavior of the most complex structural parts, mainly vaults and domes, was studied through a two-dimensional finite element analysis whose results (mostly the load transferring to the vertical elements through the support reactions) were then applied to the global simplified model (e.g., the loads on the walls).

The multi-level approach followed in this study has shown its effectiveness in the vulnerability assessment of the Castle of Melfi, pointing to possible applications to a wide variety of cultural heritage masonry buildings such as museums, castles, historic palaces, mansion buildings and holy buildings, among others. The safety evaluation of all these structures needs appropriate and extensive building knowledge, both in terms of building historic reconstruction and past restoration interventions and of material characterization. The approach was further extended to include possible structural rehabilitation techniques that follow the principles of minimum intervention that should always be followed when dealing with cultural heritage. Materials and interventions should be compatible with the architectural heritage of the castle and easily removable. This phase is typically lead by restorers, whose main interest is to preserve the original architecture and structure of the historical building. More recently, the seismic safety of historical buildings has also seen the involvement of structural engineers whose main interest is to increase the seismic safety of architectural heritage. Often times, architectural heritage is characterized by construction techniques considered dated according to today's building standards. The contrast between the needs for strengthening on one side and conservation on the other becomes evident. Minimal invasive interventions are preferred (e.g., insertion of new tie-rods, grout injections with lime mortar, local dismantling and reconstruction, mortar replacement), with the aim to improve the mechanical quality of the walls while preventing them from overturning, thus inducing a box-like behavior of the building.

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