


A knowledge-based approach for the structural assessment of cultural heritage, a case study: La Sapienza Palace in Pisa

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Abstract The full knowledge of the morphological evolution of an historical masonry building, defined more as ‘structural aggregate’ than as ‘single construction’, together with the analysis of the architectural, structural, geological and geotechnical aspects, allow the assessment of the static safety and seismic vulnerability of the complex and the design of retrofit interventions. In the present paper, a Knowledge-Based-Approach is applied to the historical building ‘Palazzo La Sapienza’ in Pisa, allowing to provide reliable results

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concerning the actual structural condition of the building avoiding the strong computational effort usually associated to the execution of refined numerical analyses. In case of complex buildings, characterized by a high heterogeneity of materials, structural typologies, geometries and so on, the adoption of a global model is not always useful to represent the effective structural behaviour. The proposed approach shows how a deep multidisciplinary knowledge of the construction can limit the use of cumbersome numerical modelling and analysis, however reaching reliable and accurate results usable also in the current practice.

Keywords Existing building · Cultural heritage · Masonry structure · In situ survey · Multidisciplinary approach · Static assessment · Vulnerability analysis

1 Introduction

The evaluation of the static safety and seismic vulnerability of existing buildings represents a significant topic for several European Countries, characterized by an architectural heritage consisting of structures not provided of suitable anti-seismic measures, with different overlapping construction techniques and often lacking of adequate maintenance. Recent seismic events in Italy (i.e., Umbria–Marche 1997, Aquila 2009, Emilia-Romagna 2012, Centre of Italy 2016) evidenced the high vulnerability of existing masonry cultural heritage and the need to define operational strategies for modifying, improving or locally retrofitting structures without altering their original characteristics and ensuring a sufficient margin of structural safety. Over the years, many of these buildings underwent changes, expansions and reconstructions transforming them from ‘single buildings’ into ‘structural aggregates’, often presenting significant decay due to insufficient maintenance, earthquakes, collapses, cracking scenarios and so on. This situation highlights the need of extensive static and seismic safety checks to plan suitable retrofit operations.

Several methodologies exist for the seismic vulnerability analysis of monumental buildings. Some of such methods (Lagomarsino 2006; Bernardini and Lagomarsino 2008) classify the buildings basing on a wide-scale damage analysis together with macroseismic intensity maps, defining vulnerability curves and identifying those structures immediately needing retrofit. Among the other approaches proposed for the structural assessment, some are based on complex models developed using the results of in situ and laser-scanner surveys and of the monitoring of the damage scenario (Barbieri et al. 2013; Chellini et al. 2014; Mallardo et al. 2008). Others, are displacement-based approaches with the definition of limit states (Lagomarsino and Cattari 2015; Novelli and D’Ayala 2015; Cattari et al.

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2015; Kržan et al. 2015). Oliveira (2003) suggested the adoption of experimental dynamic analyses to characterize the structural behaviour of existing buildings and to finally calibrate opportunely the numerical model. In all the above-mentioned methods, the existing difficulties of the numerical modelling have to be considered.

Current Italian Standard for Constructions (D.M. 14.01.2008) and the Guidelines of the Ministry for Cultural Heritage (2010) prescribe, both global checks and evaluation of the local collapse mechanisms of significant structural units, identified through the mapping of the critical points and the results of the global structural analyses. In the case of cultural heritage, the safety assessment cannot follow the usual approaches proposed by modern standards. Complex models and cumbersome numerical analyses are not always able to represent masonry aggregates: many of the structural and maintenance diseases can be fully understood only through detailed in situ surveys and the correlation between the morphological evolution of the building, its modifications throughout the centuries and its current condition. A full multi-disciplinary approach, accounting for intrinsic structural, architectural, geotechnical and functional complexities and their mutual interactions, joining historical analysis of the building's evolution, structural surveys, geotechnical investigations, mechanical characterization of materials and structural/geotechnical monitoring, is then needed as already presented by Roca et al. (2010) and Lagomarsino et al. (2014).

The present study deals with the assessment of the structural safety and seismic vulnerability of the 'Palazzo La Sapienza', in Pisa, Italy (Caprili et al. 2015a). The building, seat of the University since the fourteenth century, is currently not in use due to the pending ordinance by the Mayor's Office of May 2012 because of ensuing surveys conducted by Pisa Fire Department. The Palace is a 'structural aggregate' made up of interconnected units joined together during more than eight centuries (Caprili et al. 2015a, b, 2016).

A multidisciplinary analysis has been adopted: the deep knowledge of the aggregate and of its constitutive units/portions, the real core of the assessment procedure rather than cumbersome, complex and not fully representative global (linear and nonlinear) numerical analyses, allows the organization of the retrofit interventions preserving the original nature of the Palace and providing a sufficient margin of structural safety.

2 The adopted methodology

Modern approaches propose, also for existing buildings, Performance Based Design (PBD) methodologies based on the definition of suitable Limit States (LS), both at local and global level, to be checked by means of sophisticated nonlinear analyses. Such approaches foresee, also in the case of masonry constructions, the execution of pushover analyses based on nonlinear models of materials and/or elements. The adoption of similar methodologies need a deep knowledge of nonlinear mechanical properties of materials as well as of the structural morphology of the building, including constructive and executive details. Historical buildings consist in the aggregation, over the centuries, of several constructions, often very different for constitutive materials and adopted structural solutions, resulting in a very irregular structure both in plan and in elevation and in a great variety of masonry typologies also in the 'same' vertical element. Such irregularities make difficult accurate linear/nonlinear modelling of the whole structure and of portions thereof using traditional numerical approaches: uncertainties of mechanical properties of structural

elements, of executive details as well as of internal and external restraints would lead to not reliable results, as presented in Roca et al. (2010).

A different approach is needed to ‘disaggregate’ structural additions, intersections, stratifications and superpositions realized during the centuries and allowing analysing properties, peculiarities and deficiencies of each single original portion/unit. With the deep, aware knowledge of the construction it is then possible achieving a reliable reproduction of the construction, interpreted as the aggregation of basic structures or structural units/portions characterized by its own behaviour.

A ‘Knowledge-Based Approach’ (KBA) is hereafter proposed to show how a deep multidisciplinary knowledge of the construction can limit the use of cumbersome numerical modelling and analysis, however reaching reliable and accurate results usable also in the current practice. The so called KBA is aligned with what proposed, for example, by Lagomarsino et al. (2014) for the analysis of Palazzo del Podestà in Italy, where the decomposition analysis of the whole aggregate into different structural units, basing on the architectural and morphological evolution, is combined to the elaboration of a unique model of the aggregate, allowing to consider the two opposite conditions of full interaction and full disconnections of structural units. The KBA can be mainly summarized in the following steps:

- Step 1 Full knowledge of the building.
- Step 2 Disaggregation into structural units/portions.
- Step 3 Analysis of original single units/portions.
- Step 4 Reconstruction and analysis of the building as structural aggregate.
- Step 5 Design of retrofit interventions.

Step 1 This first phase includes both the historical analysis and the in situ investigations on the building. The complete and aware knowledge of the construction shall be the basis of any analysis executed on existing buildings. In the case of cultural heritage, historical studies completed by an accurate survey of actual architectural and structural morphology allow the recognition of the building’s evolution over the centuries up to the actual configuration. This phase leads to the detailed picture of the different construction stages and uses of the building during its working life that shall be related to the present damage due to environmental and/or human actions. In situ investigations planned to recognize constituent materials, constructive details and assess signs of the response of structural elements with relation to the different construction phases, according to the results of the historical analysis.

Step 2 On the base of the results coming from Step 1, the ‘disaggregation’ of the structure in a set of single original units/portions is developed. Each unit/portion evidences rather homogeneous constructive solutions, techniques adopted and materials’ quality (Binda et al. 2009).

Step 3 Relevant limit states (LS) associated to different levels and/or types of loading conditions shall be then recognized—if possible—within each structural unit/portion, with reference to both the global behaviour of the aggregate and the local collapse mechanisms of single structural elements or sub-portions, eventually evaluating demand and capacity. This analysis benefits of the knowledge from Steps 1–2. It is necessary to specify that, in the main context of proposed work, LS shall not be intended as the ones commonly defined by Eurocodes or Italian Standard for Constructions (D.M.14/01/2008) but as ‘critical

structural conditions' that may affect the global behaviour of the building, the safety of users or the possibility to employ the building for its main functions.

Step 4 Starting from single units/portions, the re-assembly of the structural aggregate in its actual shape is executed, determining mutual interactions and the eventual reaching of LS and critical conditions. To these purposes, numerical linear models are needed to critically analyse units/portions.

Step 5 The analyses executed in Steps 3–4 allow the selection and the design of retrofit interventions for each single structural unit/portion and for the re-assembly of the building. The KBA procedure then leads to the determination of criticisms proposing a realistic representation of structures at local level and global level (including external and internal restraints, mechanical properties of materials and structural elements, and so on), without cumbersome and often not representative linear/nonlinear models.

In the following pages, La Sapienza Palace in Pisa is analysed through the KBA, demonstrating the efficacy of the methodology for masonry structural historical aggregates.

3 Full knowledge of the building: Step 1

3.1 General description of the Palace

Figures 1 and 2 show the plan of all floors and two of the vertical cross sections resulted from the architectural survey. The building presents a trapezoidal plan (about 80.0×53.7 m) with three floors above the ground and a gable roof accessible only for maintenance. A ground-level colonnade and a first-floor arcade running roughly parallel to the ground floor sides, with maximum dimensions of 35.5 m by 21.2 m, surround the central courtyard. Due to the presence of different types vaulted surfaces and storey slabs, the height of the ground floor varies between 4.30 and 5.50 m; similar situations can be found at the first and second floors, with maximum heights respectively equal to 5.60 and 5.20 m.

The ground floor of the building houses staff rooms, university classrooms, university department offices and the historical Aula Magna. The 1st floor, besides department offices and the double volume of the Aula Magna Nuova, houses the University Library, covering about two sides of the building (along Piazza Dante square and Vicolo dell'Ulivo at 1st floor and Via della Sapienza and Vicolo dell'Ulivo at the 2nd floor), with mezzanines and loft structures to hold the books.

3.2 Critical historical analysis and building's construction phases

Constructed during the time of the Medici family over the pre-existing medieval structures, Palazzo della Sapienza is the result of modifications that brought the building to its current monumental configuration. The analysis based on the results of bibliographical and archive research enabled reconstructing the building's evolution and the different architectural arrangements. Since the late eighteenth century, the rather precarious state of the building was evidenced by the varying degrees of damage and progressive cracking scenario appeared following the changes of the functional use of the Palace.

The original medieval structures of the complex, nowadays embedded in the building's framework, included the 'Dogana del Sale', overlooking today's Via Curtatone e

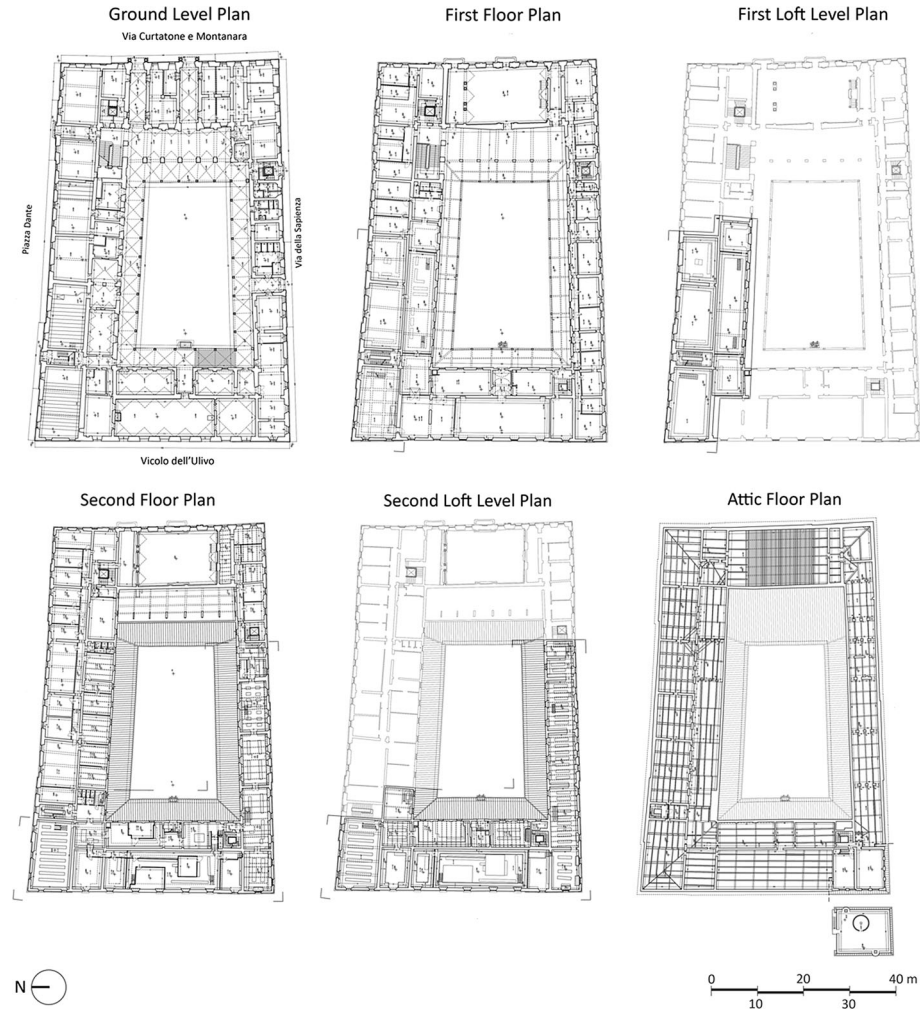


Fig. 1 Extracts of the architectural survey drawings. Plan of all floors (original scale 1:100)

Montanara, and the ‘Mercato del Grano’, situated to the west and somewhat behind (Fig. 3). The ‘Dogana del Sale’, built in 1341 (Tanfani Centofanti 1972) joining four prestigious multi-storied ‘domus’, ran along the main road axis between the Lungarno and Piazza dei Cavalieri with a structure made of stone pillars and brick masonry. The ‘Mercato del Grano’ was laid out on a quadrilateral plan with interior courtyard lined along the two long sides by porticos housing workshops of various sizes only on the ground floor. The structures on the northeast were demolished in the early twentieth century during the enlargement of the Palace, with the only exception of a tower house incorporated into the new building.

Lorenzo il Magnifico chose Piazza del Grano for the ‘Studium Pisano’, precursor of the modern university. Giuliano da Sangallo selected a spatial solution with a two-storey building surrounding a courtyard onto which opened, on the ground floor, the rooms for teaching activities and, on the first floor, the boarding school (Karwacka Codini

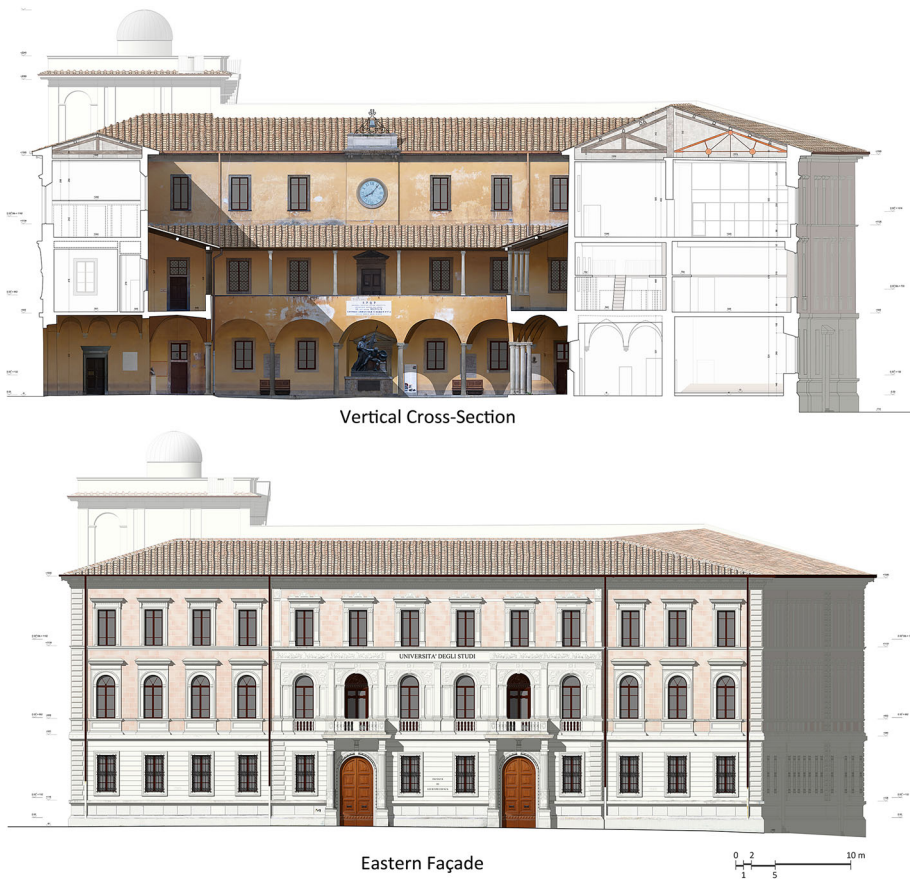


Fig. 2 Extract of the architectural survey: vertical cross-section and main façade on the east along Via Curtatone e Montanara (original scale 1:50)

2010, 2014). The shape of the free space of the atrium was resized introducing interior colonnades along the short sides of the courtyard, trying to respect the laws of perspective (Fig. 4a, b, d). In 1543, a new main entrance was opened on the southern side of the building (Lombardo 1943) adding a new façade facing the Lungarno (Fig. 4a, c).

In 1782, the Boarding School ‘Vittoriano’ and Sapienza joined by adding a storey in correspondence to the central portion of the side overlooking the actual Via Curtatone e Montanara (Fig. 5). The damage revealed to the ground floor colonnade, due to yielding of the central pillars, required the retrofit operations carried out by Francesco Bombicci and consisting of the addition of four arches supporting the vault, sustained by eight half-pillars reinforcing the old pillars and wall. In 1814, the Studio was transformed into the Imperial Academy, whose organization involved significant modifications to the building’s function and arrangement. The works, started on 1819, led to raise the western portion of the building by about 3.50 m on the outer side, by 2.40 m on the courtyard side and to the introduction of barrel vaults in correspondence of new rooms. The structural and architectural operations needed to carry out the project (i.e. closing off the interior colonnade on

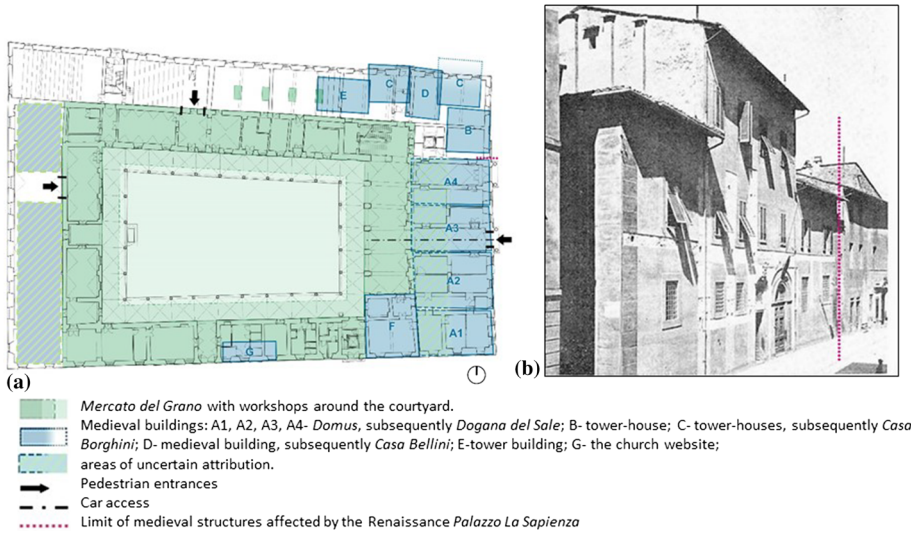


Fig. 3 a Hypothetical reconstruction of the site of the Palazzo della Sapienza as it appeared in the Middle Ages, ground floor plan; b period photo (1905) of the eastern façade along Via Curtatone e Montanara

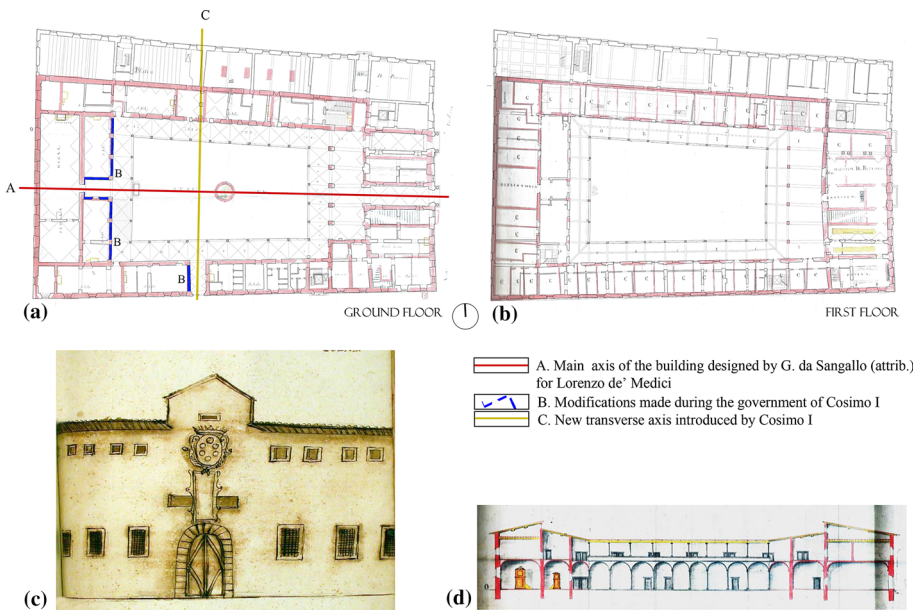


Fig. 4 Building arrangement during the Renaissance: a, b plans of the ground and first floors of the building drawn from a copy of a survey performed between 1591 and 1613 (Athenaei Pisani, Pisa, Domus Galileana, eighteenth century); c detail of the southern façade (Tronci 1643); d Athenaei Pisani, Longitudinal Section (Pisa, Domus Galileana, eighteenth century)

the western side of the first floor, demolishing various bearing walls, substituting the wooden canopies with masonry vaults, changing the arrangement, shape and size of the windows) caused considerable disruptions in widespread areas of the Palace (Fig. 6).

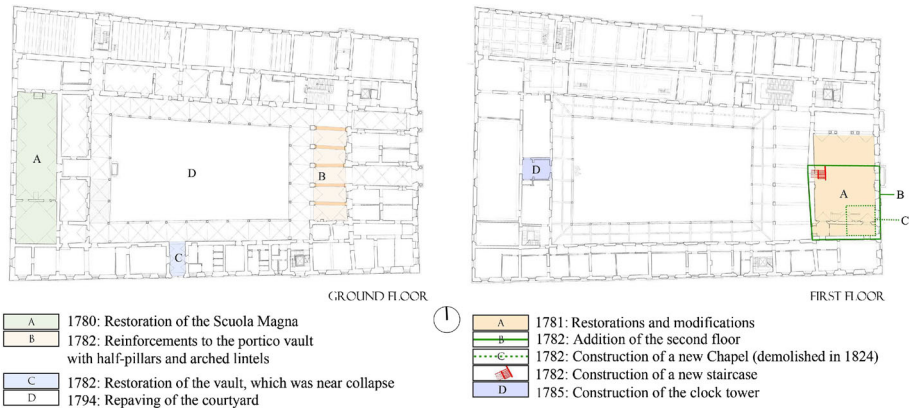


Fig. 5 Renovation, restoration and remediation works in the eighteenth centuries; ground floor and first floor



Fig. 6 Nineteenth-century works for ground and first floors

The technical surveys performed in the early nineteenth century attested the damage observed in the masonry due to water and the widespread cracking due to subsidence of the structure; further structural damages were evidenced because of the earthquake 1846. Due to the lack of funds, there was any consolidation operations—except for the introduction of five iron chains under the floor of the southern library rooms (Fig. 6).

Widespread cracking appeared at the beginning of the twentieth century, due to the raising of the roof, to the replacing of the wooden roofing with masonry vaults, to the opening of new windows and doors and to the demolition of various bearing walls. Seismic events and the destructive action on the materials of dampness and, on the building's east, of salt (Fig. 7) also influenced the structural condition of the Palace. The modifications performed on the building did not take into account its original material and morphological characteristics, forcing the Palace to sustain additional extra loads.

Towards the end of the nineteenth century, the Palace suffered a lack of space for teaching activities and for the libraries. This condition led to enlargement interventions, including the construction of a northern extension to the building, its raising up to three floors, the construction of the observatory, of a new double-volume vaulted Aula Magna on the first floor and of new staircases. The replacement of some of the damaged vaults of the interior arcade of the first floor with floor slabs of steel profiles and brick joists was also foreseen (Fig. 8).

The foundation settlement and the inadequate size of the foundations needed limiting the horizontal thrust exerted by the first and second floor vaults on the masonry bearing structures: non-thrust type floor slabs were employed, made up of steel beams, tubular bricks and steel Polonceau trusses for the roof of the new northern extension and the Aula Magna. In response to renewed need to expand the library facilities, during 1928–1929 the western building of the complex was raised to the height of the adjoining wings. From the 1930s, the Sapienza building underwent only maintenance and restoration operations, with the single exception of the vault of the new Aula Magna, rebuilt after the Second World War (Fig. 8).

3.3 Morphological and structural survey

The analysis of the morphological evolution of the Palace enabled to identify a series of critical structural issues, due to the changes made over the centuries, currently evidenced by the widespread cracking scenario in various portions of the building, to be analyzed based on in-depth knowledge of the structure. The structural complexity of the building clearly emerged from the extensive campaign of in situ studies (Caprili et al. 2015a, b), designed on the base of the results of the historical/evolutionary analysis and aiming at



Fig. 7 Areas affected by problems and structural deterioration between the eighteenth and nineteenth centuries

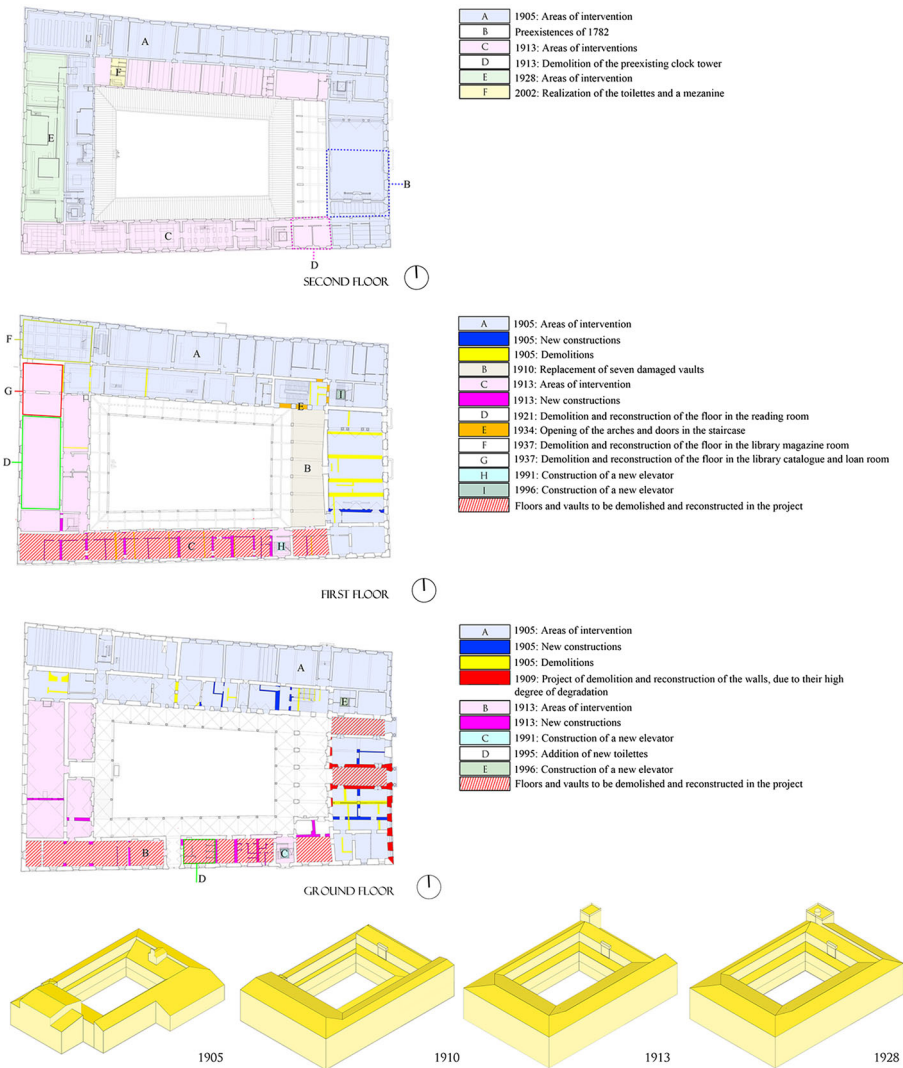


Fig. 8 Twentieth-century works: ground, first and second floors, and 3D schemes

reconstructing the current structural condition of the building (Squeglia and Bentivoglio 2015).

The masonry typologies of vertical walls were determined by removing the plaster on 50×50 cm portions with following endoscopic examination, checking the presence of cavities or adjacent facings of different thickness and type. Figure 9 shows the periods of realization of vertical walls of the ground floor (coming from the historical analysis) and the corresponding variability of masonry typologies: six different patterns and the absence of suitable connections between perpendicular walls were determined. Three tests with single and double flat jacks were performed, evidencing the uniformity of the compressive strength of the masonry and, at the same time, the high variability of the stress state in relation to the acting loads.



◀ **Fig. 9** Hypothesis of the evolutive development of the building and plan of ground floor with indications of the various types of masonry

Different types of horizontal structures are present in the building: the ground floor (Fig. 10) shows solid-brick masonry cross or cloister vaults, mainly located in correspondence of the ancient and not modified areas of the Palace (i.e., in the rooms around the internal courtyard parallel to Piazza Dante and Via dell'Ulivo, Fig. 3). In order to achieve a better estimate of their actual conditions, some masonry vaults have been checked against vertical loads by using standard tools and methods of limit analysis (Aita et al. 2012, 2015). A masonry/r.c. vault tops the double-height of the New Aula Magna. Floor slabs made of steel and brick blocks in single or double bonds (in correspondence of Piazza Dante at the ground floor, at first and second floor) were surveyed concerning structural profiles and weight of the upper filling layers.

The roofing structure, consisting of trusses of different types and materials, is the result of the modifications the building underwent over the years. On the Piazza Dante side, steel Polonceau trusses (similar to those over the double-height of the new Aula Magna, dated back to the early twentieth century) are located. The other sides of the building present wooden trusses of different sizes and shapes, with various degrees of deterioration due to the poor maintenance, bad connections among elements, presence of small solid-brick walls to support the bearing elements, accumulations of debris etc. The heterogeneity of the roof results from operations aiming at compensating for local deficiencies, finally losing sight of the building and its overall structure.

The inadequacy of the foundation structure to sustain the increasing loads due to the enlargements of the building appeared clearly since the early 1900s (Fig. 7). A deep in situ investigation of foundations was executed (Caprili et al. 2015c) through micro-core samplings at different angles in the proximity of the masonry walls, at locations planned according to the evolutionary reconstruction of the building: different types of foundations were determined (Fig. 10). The investigations performed in correspondence to the interior colonnade revealed the lack of connections between single columns, with an isolated square footing placed at -1.90 m from ground level, below the foundation of the portico pillars, laid at -1.75 m. The foundation of the portico interior walls reaches different depths (between -1.0 and -1.5 m), with width varying between 1.50 and 2.40 m. The two investigations performed along Vicolo dell'Ulivo provided similar results, with foundation depths of 2.75 and 2.10 m, and widths of 1.75 and 1.60 m, respectively. The Piazza Dante side foundation reaches a depth of about -1.50 m below with a width of 0.90 m.

The cracking scenario (Fig. 11) is the result of the modifications undergone by the Palace during the centuries, with increasing loads due to the library, to the activities developed inside the building and to the differential subsidence related to the heterogeneity of soil and foundation structure (Figs. 7, 10). The progressive growth of the cracks, especially in proximity of the different structural units, is still quite evident in the building's vertical and horizontal surfaces. A wide cracking scenario is present in the corner between Via Curtatone e Montanara and Via della Sapienza, due the incorporation of ancient tower houses into the current structure, and in correspondence of the vaulted surfaces of colonnade, characterized by the evidence of differential settlements and by structural problems already identified during the 18th and 19th centuries (Fig. 10).

Two monitoring systems (one through periodical readings of fixed steel reference gauges and one through continuous readings at electronic transducers) were set up to determine the ongoing subsidence and the growth of the damage (Fig. 10). The analysis of

Fig. 10 Distribution of the floor slabs and types on the building's ground floor, Foundation plan, different typologies evidenced and ground floor plan with indication of monitoring points (continuous and periodic systems)





Fig. 11 Description of the cracking scenario at the different floors of the building

the monitoring results enabled the identification of the differential displacements, highlighting the evolution of structural cracking mainly in correspondence of the vaults of the ground floor colonnade and first floor arcade.

3.4 Ground investigation

An extensive site investigation campaign allowed a careful assessment of the soil stratigraphy below the palace, and of the geotechnical parameters and properties characterizing the foundation soil. The ground investigation included 7 continuous core drillings, 6 static penetration tests (CPTU), 2 tests with seismic dilatometer (SDMT) and 1 dynamic penetration test (DPSH) (Caprili et al. 2015c).

It was so possible to perform the site seismic response analysis to define the effective response spectrum for the considered building and analysis. The dynamic characterization of the foundation soil allowed the execution of a local seismic response analysis, necessary

for the safety assessment of the building. The definition of the seismic input was performed by selecting 7 compatible accelerograms and applying them directly to the representative multi-layer model of the subsoil of the site on which the building is located. The site response analysis was carried out in a free-field situation (i.e. neglecting the presence of the building) and in one-dimensional conditions, assumptions justified by the horizontal alignment of all the soil layers (Iervolino et al. 2014; Caprili et al. 2015c). The so obtained elastic response spectrum (Fig. 12) directly takes into consideration the geotechnical characteristics of the subsoil of the site on which the analyzed building is realized; amplification due to the presence of soft soil layers are then directly included. The shear wave velocity up to 40 m was measured by means of the Down Hole test and of six resonant column laboratory tests, analysing the change of stiffness and damping of soil with deformation increment. The subsoil model (Fig. 12a) was processed using the software STRATA (Kottke and Rathje 2008) accounting for soil non-linearity. Figure 12b shows the elastic response spectrum for the site under examination (ground type C) in comparison to the standard spectrum for a return period of 712 years.

4 Disaggregation into structural units/portions: Step 2

Critical issues of the structural aggregate were identified with reference to the single structural units recognized as constitutive parts of the building. The Palace was considered as composed by a suitable number of units connected by horizontal structures (foundations, floors and vaults, roof) influencing the mutual interactions among portions, preventing or activating local mechanisms. Structural units were determined in relation to the morphological/structural evolution of the Palace, to the modifications undergone during the centuries, to the presence/absence of connections among vertical and horizontal elements and to the interpretation of the cracking scenario. The behaviour of the units was investigated firstly separately and then in the framework of the aggregate itself.

A diffused lack of connection between masonry walls and different masonry typologies (even in the same element) are evident and related to the evolution of the Palace: Fig. 13, for example, show the presence of masonry arches of ancient openings nowadays closed

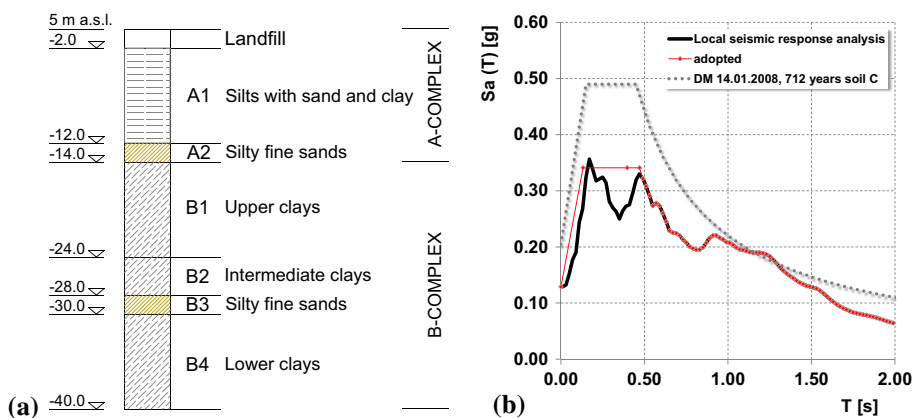


Fig. 12 Elastic response spectrum results for the site in comparison to regulatory indications for type C soils



Fig. 13 Discontinuities of vertical masonry walls: absence of connection between perpendicular walls, arches identified in correspondence of the first floor level, probably old windows currently closed

due to the modified functional needs. The heterogeneity of masonry walls is higher at the ground and first floors (i.e. corner Via Curtatone e Montanara/Via della Sapienza, Fig. 9) respect to the second level, realized at the beginning of the nineteenth century (Fig. 8).

Horizontal floors and roofing systems can allow the connection among masonry walls, preventing the activation of local mechanisms; the horizontal systems of Palazzo La Sapienza present vaulted surfaces (mainly at the ground floor where no significant changes took place) and more recent floors with steel profiles (Fig. 10). If vaulted structures can constitute a rather significant restraint for vertical structural elements, horizontal steel floors cannot be considered a ‘rigid’ diaphragm.

The considerable geometric heterogeneity of foundations also influence the mechanical response of the building and its bearing capacity in relation to the variable stiffness of the soil.

The above-described more or less rigid systems connect the different structural units coming from modifications of the Palace executed during the centuries. According to the results of structural/architectural/geotechnical surveys, the following systems are recognized as different Structural Units (US) inside the aggregate (Fig. 14).

US1. Corner between Via Curtatone e Montanara and Via della Sapienza.

US2. Open gallery and arcade at the ground floor and colonnade at the first floor.

US3. Aula Magna and adjacent rooms at ground floor.

US4. Double volume of the Aula Magna Nuova at the first/second floor.

US5. Portion of the building facing Piazza Dante.

US6. Portion parallel to Piazza Dante facing the internal courtyard (ground and first floors).

US7. Second floor of the Palace and roof.

US1. *Corner between Via Curtatone e Montanara and Via della Sapienza.* As visible from Fig. 14, pillars characterized by the presence of full bricks and squared stones demonstrate the pre-existence of ancient tower-houses incorporated and connected to adjacent masonry structures during the centuries

US2. *Open gallery and arcade at the ground floor and colonnade at the first floor.* The internal arcade and the open gallery facing the entrance along Via Curtatone e Montanara at ground floor present vaulted cross surfaces with different geometries and heights

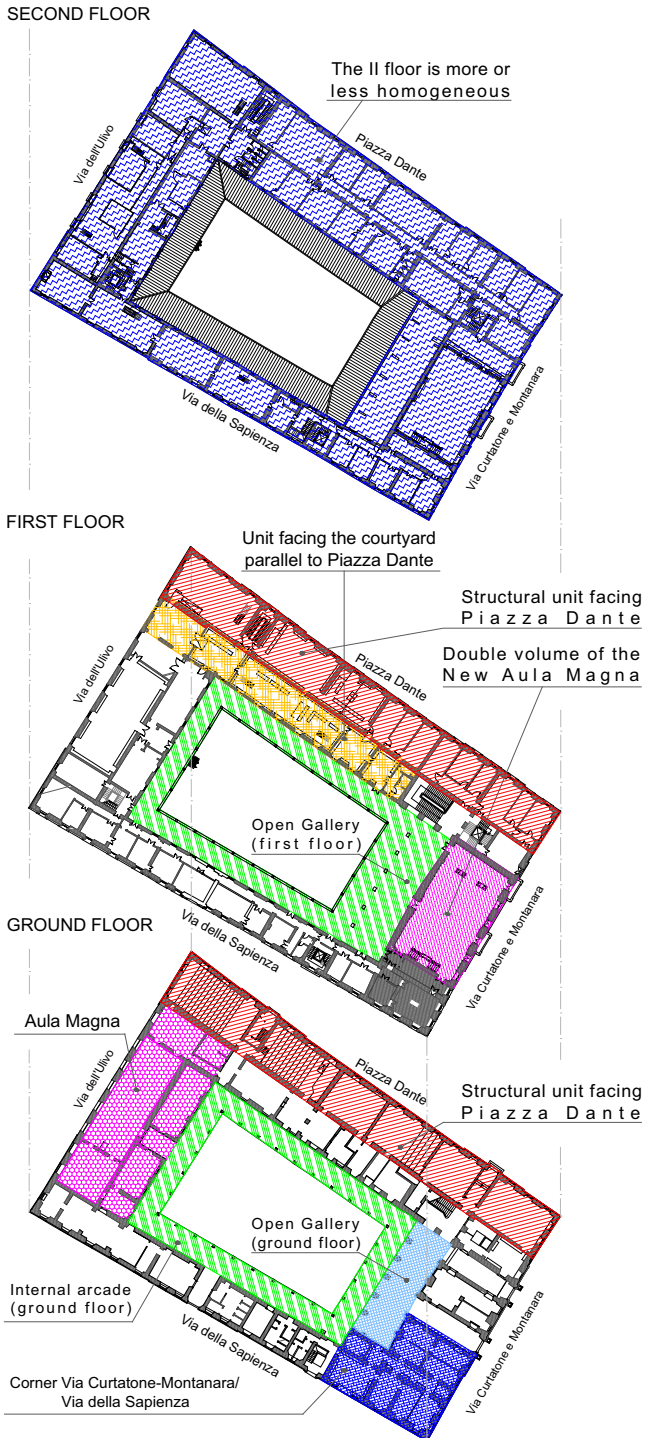


Fig. 14 Structural units that can be determined as portions of the aggregate

- US3. *Aula Magna and adjacent rooms at ground floor.* The portion of the building at the ground floor facing Via dell'Ulivo houses the historical Aula Magna and additional rooms once used for teaching activities. The Aula Magna and the adjacent room, actually characterized by the presence of a partition wall, present the same vaulted surface on which the masonry wall at the first floor directly loads (Fig. 4). The two teaching rooms facing the courtyard are the result of the closure of the original lodge that, on the western part of the Palace, exactly reproduced the opposite situation of the arcade, as justified by ancient plans and through the analysis of the cracking scenario and from the observation of the masonry pillars of the original colonnade (Fig. 9)
- US4. *Double volume of the Aula Magna Nuova at the first/second floor.* The double volume of the Aula Magna Nuova, born at the end of the nineteenth century due to the modifications of the building to overcome the new use requirements, has an internal open space not provided by transversal retaining walls (demolished in 1905, Fig. 8)
- US5. *Portion of the building facing Piazza Dante.* The block constituted by the rooms facing Piazza Dante constitutes a separate structural unit, generated in a more recent period (Figs. 3, 4), modified at the beginning of the twentieth century and clearly distinguishable from the rest of the building due to the presence of horizontal steel storey slabs and of masonry walls not connected to the pre-existences. This situation is clearly visible at both the ground and the first floor
- US6. *Portion parallel to Piazza Dante facing the internal courtyard.* The architectural and structural surveys of the building evidenced the lack of perpendicular walls in correspondence of the first floor of the building in the portion facing the internal courtyard and parallel to Piazza Dante. The increasing need of place for the library and the new activities developed in the building led to the demolition of the walls perpendicular to the internal main façade (Fig. 6)
- US7. *Second floor of the Palace.* The second floor of the building, according to the morphological and structural analyses of the Palace, can be considered a 'separate' structural unit (Fig. 4), characterized by a more recent edification and a more homogeneity for what concerns materials, masonry organization and floor typologies

5 Analysis of structural units/portions: Step 3

The structural behaviour of the US of the building (Fig. 14) was studied identifying local 'vulnerabilities' associated to LS and local 'criticisms' related to single structural/not structural elements and to local peculiarities.

Vulnerabilities were analysed through the adoption of local models able to reproduce the main features on the base of the knowledge obtained from in situ and experimental investigations. The influence of local critical mechanisms, on the other hand, cannot be easily evaluated by means of models and analyses, nevertheless such mechanisms can be relevant for the static and seismic safety. With reference to the above list on Structural Units (US), the following vulnerabilities and critical points/aspects are recognized.

US1. Corner between Via Curtatone e Montanara and Via della Sapienza

The nature of masonry vertical walls cannot allow achieving a thorough assessment of the building's current margin of safety. The widespread discontinuities in the structure's geometry, materials and constraints, the presence of voids, air spaces and facings, together with the dubious degree of collaboration, lead to local stress concentrations, confirmed by the diffused cracking scenario revealed during in situ surveys and by the results of the experimental flat jack tests.

The discontinuity and heterogeneity of masonry for their respectively different construction typologies, realized in different periods with lack of mutual connections, yields to the development and progressive spreading of a diffused cracking scenario visible both in vertical and horizontal structures, allowing the clear recognition of US1 inside the aggregate (Fig. 15).

The cracking scenario in the vertical masonry walls relates to the differential subsidence of foundations: as visible from Fig. 10, in correspondence of US1 several typologies of foundation structures (i.e. Type 1, Type 2, columns and pillars) are recognized, characterized by different properties in terms of geometry and stiffness (Table 1). The heterogeneity of foundations strongly affects the relative displacements between the different US of the structural aggregate, with following spread of the cracks.

US2. Open gallery and internal arcade at the ground floor and colonnade at the 1st floor

The diffused cracking scenario revealed in the crossed vaulted surfaces of the ground floor is the direct consequence of the differential subsidence due to the heterogeneity of foundations (Fig. 10; Table 1). The damages due to such discontinuities were highlighted by past seismic events (Fig. 7).

As visible from the in situ investigations, the foundation structures of the columns of the internal arcade are disconnected to one another and lay at different levels respect to the masonry walls facing the courtyard. The absence of horizontal connection between the pillars of the internal arcade make the columns behave 'independently', with relative displacements and rotations. These differences result in differential subsidences evidencing the non-uniformity of the mechanical response of the foundation system to the acting loads, the variability of soil equivalent stiffness and of ultimate loads.

The analysis of the structural condition of foundations, considering actual loads (permanent loads due to structural elements—masonry walls, horizontal floors, vaulted surfaces—as well as live loads and seismic action) allowed the determination of the unit load-

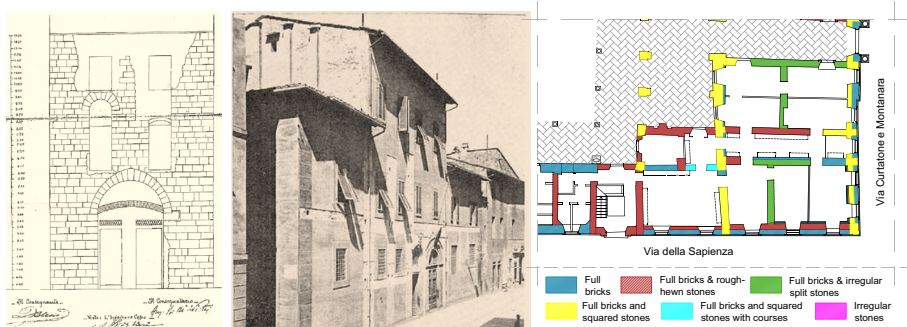


Fig. 15 US1—corner between Via Curtatone e Montanara and Via della Sapienza, heterogeneity of masonry vertical walls and incorporation of tower houses

Table 1 Initial stiffness evaluated through the elasticity theory for the different investigated sections, main dimensions of the different foundation typologies and ultimate load (Brinch-Hansen formulation)

| Foundation type | K (kPa/m) | B (m) | A (m ²) | q_{lim} (kPa) |
|---|-----------|-------|---------------------|-----------------|
| Colonnade | 32,560 | – | 1.21 | 2700 |
| Portico pillars | 25,300 | – | 1.80 | 2250 |
| Via Curtatone Montanara/della Sapienza (Type 2) | 3200 | 2.4 | – | 1900 |
| Piazza Dante/Via Curtatone e Montanara (Type 3) | 7600 | 0.9 | – | 1650 |
| Other (Type 1) | 4500 | 1.6 | – | 1270–3000 |

subsidence diagrams for the foundation-ground assemblage of the various analysed sections (Fig. 17a). The responses of the foundation elements evidence, especially for unit load values below 1.0 MPa, a wide variability. The substantial difference in the behaviour of the building's perimeter and of its interior (i.e. colonnade and pillars of the interior courtyard) leads to considerable damage in the areas where variation of stiffness characteristics and of acting loads are higher (Fig. 17b). The situation of foundation settlements, the different periods of realization and the following absence of connections between vertical elements also influence the behaviour of the colonnade at the first floor: relative displacements between the wooden roof of the colonnade and the masonry walls are clearly visible (Fig. 16). In particular, significant slips were determined between the wooden elements and the masonry walls: this was due to the insufficient support length of the wooden joist inside the walls (in some cases lower than 2 cm) and, together, to the thermal daily variations affecting the building as proved by the analysis of monitoring continuous data. The relative movements between the colonnade and the masonry walls are visible both at the ground floor (where vaulted surfaces are present) through the survey of the cracking scenario and at the second floor. Different foundations systems are, in fact, present, as revealed by the in situ investigations, and the differential displacements are evident considering the structural response of foundations (Fig. 17).

US3. Aula Magna and adjacent rooms

The progressive growth of the building at the first floor, necessary to house the library and its volumes, led to the introduction of a solid brick curtain wall dividing the library rooms from the ones used for teaching activities. The masonry wall—with thickness equal to 17 cm, height equal to two floors—already present at the end of the sixteenth century

**Fig. 16** Internal colonnade (1st floor): relative displacements between wooden elements and masonry walls

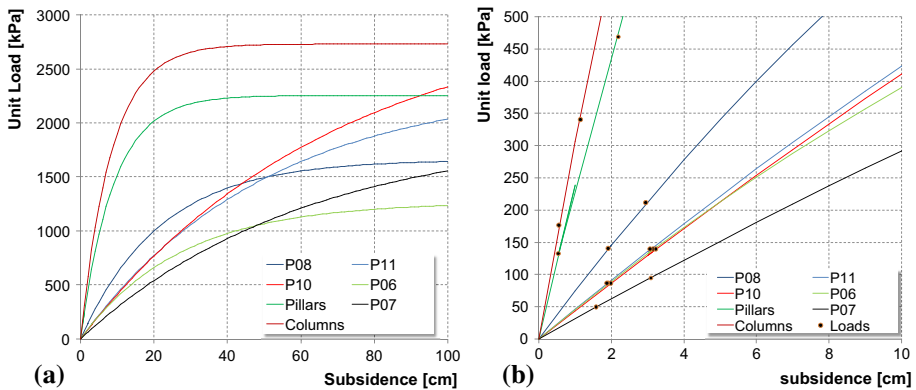


Fig. 17 **a** Unit load-subsidence curves for the different sections investigated, **b** estimates of the subsidence of different building zones as a function of the applied loads

nowadays represents the effective separation between the portion of the building belonging to the University of Pisa and the University Library (Fig. 4). The wall directly rests upon the vaulted surface of the room adjacent to the Aula Magna, generating a critical situation worsened by the eventual presence of seismic action.

The analysis of the cracking scenario affecting the Aula Magna at the ground floor, the adjacent rooms and the other areas facing the internal courtyard (Fig. 13) depends on the progressive modifications executed during the centuries, as highlighted by the historical analysis. Severe damages are present in correspondence of the vertical masonry walls and of the vaulted surfaces of the two rooms facing the internal court. This portion was primarily part of the colonnade (Figs. 4, 9) and the closure of the area dated around the end of the XVI century; the relative displacements between the colonnade and the vertical masonry walls of the building—proved by the analysis of the monitoring data—are connected to the evidenced cracking scenario.

US4. Aula Magna Nuova: double volume at first and second floor

The analysis of the double volume (Fig. 6) of the New Aula Magna evidenced cracks in the vertical masonry walls (not perfectly visible due to presence of ancient paintings) and on the vaulted surface. The in situ survey of the roofing system highlighted the presence of steel Polonceau trusses—date back to the early twentieth century—sustaining the roof; the vaulted surface, on the other hand, is made up of thin brick elements (3.0 cm) between r.c./masonry trusses used to prevent horizontal thrusts on the masonry walls (Fig. 22). The cracks visible in correspondence of the vaulted surface are, mostly, due to the ‘geometrical’ distance of light brick elements; the support of those elements in correspondence of the main bearing truss is equal, at maximum, to 3.0 cm. Such situation generates the possible detachment and the following downfall of the elements, with risk to people and to the structure itself.

An additional vulnerability determined in US4 relates to the possible activation of overturning mechanisms in correspondence of the two main longitudinal walls (i.e. the external façade and the corresponding parallel internal wall). The demolition of the internal masonry restraints, due to the increasing need of space to house the activities developed in the building exposes the Palace to local collapse mechanisms. The lack of horizontal restraints between the two opposite facades and the inability of the vaulted surface to

provide a rigid diaphragm action (no steel chains are present) emphasizes the above-mentioned problem. The linear kinematic approach allows analyzing the overturning phenomena of external and internal walls: the evaluated PGA activating the mechanism is equal to 0.112 g respect the design value of 0.118 g.

US5. Portion of the building facing Piazza Dante

US5 is easy to be determined in the global context of the structural aggregate of Palazzo La Sapienza due to both the plan organization of the ground floor (primarily only one row of rooms was present, Figs. 3, 4) and the structural features (i.e. the slabs made up of steel profiles and brick elements, the masonry typologies). More recently realized, US5 shows a higher homogeneity respect to the rest of the structural aggregate, from which it is ‘separated’ as evidenced by the lack of connection between perpendicular walls.

The high level of knowledge obtained concerning the horizontal storey slabs through the structural survey (with determination of the profiles, thickness of filling materials and so on) together with mechanical characterization of steel properties, allowed to execute the safety checks both in terms of strength and deformability. Taking into consideration the effective loads acting on the horizontal structures (in relation to developed activities), safety assessments according to Italian National Standard for Constructions (D.M. 14/01/2008) provided good results, except in one isolated case (room P2-52, second floor).

US6. Portion parallel to Piazza Dante facing the internal courtyard

The progressive increase of space due to the need to house the libraries—executed through the demolition of internal walls and the introduction of mezzanines at the first and second floors—led to overload the bearing structures. This condition strongly alters the original local response of masonry walls towards dynamic loads, exposing the building to out-of-plane overturning mechanisms where the mezzanines’ cantilever beams are anchored without introducing adequate restraints between perpendicular walls.

The out-of-plane problems are evident in correspondence of the sub-unit facing the internal courtyard and parallel to Piazza Dante, resulting from the modifications described in the historical analysis (Figs. 6, 8). Since typical mechanisms can be recognized, the linear kinematic analysis can be adopted to determine the value of horizontal peak ground acceleration (PGA) activating the mechanism. As an example, the lack of perpendicular retains in correspondence of the internal wall results in the possible activation of the global overturning of the wall, both at first and second level. The PGA related to the overturning mechanism are respectively equal to 0.060 and 0.079 g for the first and the second floor, towards a foreseen design PGA equal, for the considered site, to 0.118 g.

US7. Second floor of the building and roof

The second floor of the building, more recently realized, is homogeneous for what concerns both the masonry typologies and horizontal storey slabs (made up of steel profiles with lightening brick elements). The detailed survey of the second floor of the building and of the roof highlighted several deficiencies and criticisms that, even if not directly quantifiable, shall be taken into consideration for both the vulnerability assessment and the eventual retrofit.

All the lightening brick elements of the storeys are characterized by a brittle behaviour with possible collapse due to the presence of punctual loads and to the lack of an adequate distribution slab. Concentrated loads depends on infill masonry panels loading upon the brick elements: despite the satisfaction of strength and deformability requirements by steel profiles, this situation represents a relevant problem for the safety of the building and of the

users. An analogous condition has been revealed in correspondence of the storey slabs of the gable roof, where several debris are stored as a consequence of past interventions, often loading upon the lightening elements.

The roof structure presents diffused maintenance problems (Fig. 18): the wooden elements, subjected to specific investigations aiming to determine the damage level due to humidity, insect, insufficient maintenance, are affected by a high degree of degradation. Several criticisms are present both in the primary and in the secondary elements; several other wooden beams are not able to satisfy the safety checks foreseen by actual standards for constructions in terms of strength and deformability. The structural survey of the roof system also evidenced significant cracks in correspondence of the support of wooden trusses on the masonry walls and on the eaves (more recently realized), due to the horizontal thrusts of the trusses themselves. Connection between elements (both steel and wooden ones) are not correctly realized and need to be improved to globally increase the safety level. As an example, it is necessary to remove brick elements sustaining the wooden beams directly loading on the storey slab, to replace existing connections with more safety and efficient systems and so on.

Table 2 briefly summarizes the critical situations revealed in the different structural units present in Palazzo La Sapienza.



Fig. 18 Degradation and problems affecting the roof structure of the Palace

Table 2 summary of criticisms, vulnerability and structural problems revealed in Palazzo La Sapienza

| Structural unit | Criticisms, vulnerabilities, structural problems |
|--|--|
| US1 <i>Corner Via Curtatone e Montanara/Via della Sapienza</i> | Local stress concentrations evidenced by the diffuse cracking scenario in correspondence of discontinuities of materials/elements Differential subsidence of foundations due to their heterogeneity |
| US2 <i>Open gallery and internal arcade at the ground floor and colonnade at the first floor</i> | Differential subsidence of foundations between colonnade and internal walls due to heterogeneity of foundations and lack of connection between columns' foundation structures Relative displacements of wooden elements due to insufficient support length and thermal daily variations |
| US3 <i>Aula Magna and adjacent rooms</i> | Local stress concentrations evidenced by the diffuse cracking scenario in correspondence of the walls closure of the original lodge Masonry wall (separating University and Library) directly loading upon the vaulted surface of the ground floor |
| US4 <i>Aula Magna Nuova: double volume at first and second floor</i> | Possible cracking and collapse of lightening brittle brick elements of the vaulted surface due to insufficient support length Possible activation of out-of-plane overturning mechanisms due to the lack of transversal restraints |
| US5 <i>Portion of the building facing Piazza Dante</i> | Disconnection from the rest of the structural aggregate |
| US6 <i>Portion parallel to Piazza Dante facing the internal courtyard</i> | Possible activation of out-of-plane overturning mechanisms due to the lack of transversal restraints |
| US7 <i>Second floor of the building and roof structure</i> | Possible cracking and collapse of lightening brittle brick elements of storeys due to the presence of concentrated loads Diffuse lack of maintenance and damage of wooden elements of the roof Not adequate connection among the elements of the roof |

6 Structural behaviour of the aggregate: Step 4

In the case of historical buildings, mainly considered as 'structural aggregate' instead of single construction, the elaboration of a nonlinear global numerical model cannot be able to provide reliable results and usually results in a strong computational effort. This is due to the difficulty in representing the heterogeneity of the different structural units and their mutual interrelationships. These last ones are, at the same time, responsible of the inadequacy of independent numerical models of the single US, whose interaction finally determines the global response of the structural aggregate.

The adoption of linear schematizations, on the other hand, can lead to safety assessment often too conservative respect to reality, not always compatible with the need to preserve the historical building and to increase the safety margin towards seismic action.

In the present work, the choice of adopting a linear model—despite its simplifications—was mainly due to two main reasons: first, the elaboration of a linear model is needed to

execute safety checks towards static loads—determining static deficiencies and criticisms on of masonry vertical walls, horizontal stories and roof elements. Secondly, nonlinear models and analyses require a very deep knowledge of masonry materials to provide reliable results. In the case of Palazzo La Sapienza, in situ investigations highlighted the presence of material discontinuities, intersections and superposition, presence of arches/windows/doors closed during the centuries and so on, leading to a very different modelling of the real effective nature of bearing elements.

The same considerations are valid for the modelling of connections among perpendicular walls: single structural units can be determined basing on the historical and morphological evolution but the connections among them are not always clear, resulting in the need to make simplifications and assumptions. The main aim of the FEM model realized in SAP 2000 consists then in providing reliable indications about the stress state of masonry walls under static conditions, the global estimation of the behavior of the Palace considered as an aggregate and the weak zones with undesirable behaviour (Meli and Pena 2005; Roca et al. 2010; Lagomarsino et al. 2014). In such condition, considering the approximation made in representing such variability and heterogeneity of materials, the adoption of linear models is acceptable as preliminary tool, usually providing conservative indications.

The ‘guided’ interpretation of obtained results can provide safe suggestions for engineers and practitioners who are not so addicted with nonlinear methodologies and approaches.

The KBA then allows achieving an increased margin of safety together with the preservation of the monumental nature of the building complex through a rational planning of retrofit interventions.

The analysis of the single US allowed the determination of criticisms and vulnerabilities characterizing Palazzo La Sapienza considered as ‘structural aggregate’. On the base of such critical analyses, it is possible ‘re-aggregating’ and critically investigating the monumental building adopting a global model. In such case the model, even if representative of an enhanced condition respect to the current one (i.e. connection among perpendicular walls, homogeneity of masonry typology in the single vertical wall, absence of cracking scenario, internal and external restraints and so on), allows to numerically validate the results of the KBA. The analysis of the re-aggregated structure provides reliable results concerning the static behaviour of the Palace, and useful indications about the effects of horizontal actions.

Beside what presented, the elaboration of a global three-dimensional model is nowadays required in Italy by D.M.14/01/2008, for the execution of safety checks on masonry vertical walls, horizontal floors, vaulted surface and roof elements. The global model represents an improved condition respect to the effective situation of the Palace. A simplified FEM model of the building was realized using SAP 2000, with two-dimensional elements for masonry walls and vaulted surfaces modelled as equivalent plane elements (Lagomarsino 2006) and one-dimensional elements for the steel profiles of the floor slabs, the roofing and the university library mezzanines (Fig. 19). The vaulted (cross/barrel with lunettes) surfaces were modelled adopting isotropic or orthotropic membrane elements. The elastic bi-dimensional shell element (membrane and flexural stiffness) describing the floor is identified by one or two principal directions, with specific Young and shear moduli. An ideal equivalence is set between the typology of vault examined and the plane element, having the same planimetric dimension, thickness and material (characterized by the elastic moduli previously defined according to stiffness equivalence principles). An isotropic plane element is accounted for if the vault typology is symmetrical (e.g. cloister and cross vault) and an orthotropic one if unsymmetrical (e.g. barrel vault). The equivalent

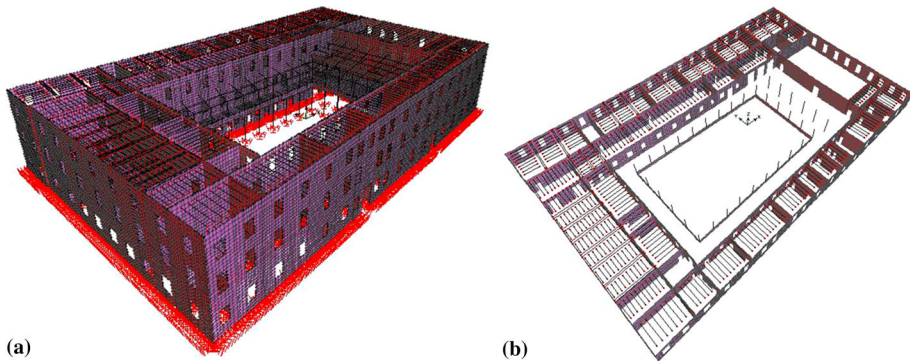


Fig. 19 **a** Global numerical model of the building, **b** example modelling of the first floor slab

parameters, describing the vault stiffness and derived by elastic analyses, represent an overestimation of the actual properties, because of probable existing damage of material degradation, often surveyed in ancient masonry structures. This leads to the equivalence to an isotropic plane element, if the vault typology is symmetrical (e.g. cloister and cross vault), and to an orthotropic one, if unsymmetrical (e.g. barrel vault).

The Winkler model allows representing the soil-structure interaction, with elastic three-directional springs calibrated according to the results of the geotechnical investigations (Table 1) and in relation of foundations' shape and size (Fig. 10). The mechanical characteristics of the masonry material were selected according to the results of in situ investigations, flat-jack tests and to the indications contained in the Ministerial Memorandum Circ. 617/2009, for the different types of masonry patterns (Table 3). The reduction of the stiffness values up to 80 and 60% of the undamaged condition (respectively on the ground and the first and second floors) simulates cracking phenomena (Paulay and Priestley 1991).

Modal Response Spectrum Analysis (MRSa) (Fig. 12) with behaviour factor equal to 2.25 was performed. The value of the q -factor was selected on the base of what reported on the Italian Standard for Constructions (D.M.14/01/2008), prescribing a value equal to 1.5 α_w/α_1 for existing masonry buildings not regular in elevation, being, without specific investigations, α_w/α_1 equal to 1.50. The assumed $q = 2.25$ is conservative with respect to the prescriptions of the Guidelines provided by the Ministry for Cultural Heritage (2011) specifying a maximum value up to 2.80.

Table 3 Strength and elastic moduli values (not cracked) adopted for the different masonry patterns

| Masonry | σ (MPa) | E (MPa) | Notes |
|--|----------------|---------|---------------------------------|
| 1. Full bricks | 2.25 (*) | 1500 | * flat jack tests (mean values) |
| 2. Irregular split stones | 1.40 | 870 | |
| 3. Full bricks and rough-hewn stones | 2.20 (*) | 1500 | * from flat jack tests |
| 4. Full bricks and irregular split stones | 2.20 (*) | 1500 | * type similar to (3) |
| 5. Full bricks and squared stones | 2.20 (*) | 2150 | * type similar to (3) |
| 5. Full bricks and squared stones with courses | 2.64 (*) | 1500 | * (3) improved by courses |

The response spectrum obtained from a local analysis was ‘adapted’ to the actual shape of standard one, with a ‘linearized’ plateau for periods lower than 0.50 s, neglecting minimum peaks of spectral acceleration for sake of caution. Table 4 summarizes the results of the modal analysis. Global vibration modes were determined for both flexural behaviour in x and y directions; the significant periods ranged between 0.89 s (global flexural mode in y direction) and 0.59 s. Looking together the vibration periods of Table 4 and the spectral accelerations presented in Fig. 12, it is strongly evident the importance of executing a local response seismic analysis for the evaluation of action and following solicitations on historical buildings.

The safety checks were executed following D.M.14/01/2008 and Circ. n°617/2009. In particular, D.M.14/01/2008 and the Guidelines of the Ministry of Culture and Heritage (2011) prescribe the execution of safety checks including both global and local verifications of single bearing elements and of structural sub-portions for which the activation of local collapse failures (i.e. overturning of corners, of facades, of galleries and others) has been highlighted by the survey of the actual in situ cracking scenarios, but the evolutive analysis and so on. Global safety checks on existing masonry walls included verifications towards in- and out-of-plane flexural mechanisms and shear actions.

Results confirmed the vulnerabilities and the criticisms determined through the KBA described in the previous paragraphs, as briefly presented below.

The global checks on masonry walls evidenced that the building essentially satisfies the safety requirements with regard to both static and seismic combinations, with the exception of extremely small-sized, slender elements characterized by a considerable heterogeneity in shape and materials and by extensive cracking due to the progressive structural layering and modifications made to the building. Relatively critical conditions were encountered in the area of the historical Aula Magna (US3), where two of the original spaces of the colonnade were closed-up, and in the proximity of the double volume of the new Aula Magna (US4).

Problems of out-of-plane overturning were found in correspondence to the interior wall of the first floor towards the arcade parallel to Piazza Dante (US6), where the perpendicular masonry restraints were removed without taking into account their structural effects, and in correspondence to the wall of the double volume of the new Aula Magna (US4).

The wooden roofing structures (US7) presented signs of widespread degradation due insect and mold damage, with consequent reduction of the resistant section of the bearing elements, with the following not satisfaction of the prescribed safety requirements. Other critical issues were linked to the presence of curtain walls supporting trusses, supports of

Table 4 Vibration periods and modal participating masses for significant modes

| Mode | Period (s) | M _x (%) | M _y (%) | M _{x,tot} (%) | M _{y,tot} (%) |
|------|------------|--------------------|--------------------|------------------------|------------------------|
| 18 | 0.8963 | 0 | 67 | 1 | 68 |
| 28 | 0.8121 | 25 | 0 | 37 | 69 |
| 29 | 0.8020 | 5 | 0 | 43 | 69 |
| 30 | 0.7964 | 5 | 0 | 47 | 69 |
| 31 | 0.7956 | 5 | 0 | 52 | 69 |
| 33 | 0.7866 | 19 | 0 | 71 | 69 |
| 41 | 0.7467 | 3 | 11 | 80 | 81 |
| 85 | 0.5974 | 1 | 5 | 90 | 93 |

varying shapes and types, and to the accumulation of construction debris in various areas of the garret.

The horizontal structures (floors), with the exception of one under-dimensioned slab on the building's second floor, did not evidence significant structural deficiencies in terms of either resistance or deformability: they were realized relatively recently (during the post-war restructuring of the building). Analogous considerations can be executed for the vaulted structures, which, though presenting widespread cracking, especially in the colonnade area and in the proximity of the original Aula Magna, were able to sustain a load equal to 350 daN/m^3 , as demonstrated by the in situ load tests executed.

Analysis and safety checks of the foundation structures revealed the substantial heterogeneity of the overall mechanical behaviour of the foundations, variable stiffness of the ground-foundation assemblage and different limit load values, minimal for ribbon foundations (i.e. masonry walls) and higher in the case of isolated foundations (below US2). The unit load-subsidence diagrams for the foundation-ground assemblage for the various analysed sections highlight the different responses of the various foundation elements as a function of the geometry, especially for unit load values below 1.0 MPa. A substantial difference in the behaviour was visible between the building's perimeter and its interior part (i.e. colonnade and pillars of the interior courtyard, Fig. 17b); in the presence of either static or seismic loads, these local differences can expose the building to considerable damage in those areas with significant differences in rigidity characteristics or those subject to more heterogeneous loads.

7 Retrofit of the structural aggregate: Step 5

Actual standards provide indications and methodologies for the global retrofit of existing buildings. Such approaches, together with a significant increase of the structural safety, often lead to the modification of the resistant system of the construction: this is what happens, for example, with the introduction of bracing steel systems, of dissipative devices, of r.c. walls and, in relation to the numerosity, in the case of diffused local retrofit interventions.

The adoption of a 'global' approach for the selection and the design (both conceptual and executive) of the retrofit of an historical masonry building can be not adequate to preserve its original nature and configuration, made up of several different but interconnected units. The retrofit shall be able to preserve the traditional aspects of the building allowing the increase of the safety level.

The determination of local criticisms and vulnerabilities, following disaggregation/re-aggregation of the building and KBA, allows organizing the retrofit in the form of 'superposition of local interventions' concerning the different structural units, globally allowing the improvement of structural performance in terms of strength, stiffness and ductility. In this sense, retrofit is the natural extension of the structural analysis based on the historical investigation of the cultural heritage, on its morphological evolution and of the following determination of criticisms, problems and vulnerabilities due to insufficient maintenance, modifications, changed uses and so on.

The retrofit of Palazzo La Sapienza lies in the main framework of the architectural, structural, energy and equipment improvement of the building, designed to house libraries, departments and teaching rooms for the University. The new use and architectural features of the building, together with the need to make it the central point of the University and of

the urban context of Pisa, led to the design of retrofit intervention for several existing elements and portions. Retrofit interventions have the primary objective not altering the original configuration of the aggregate, taking into consideration the results of historical and in situ investigations. In this sense, the results coming from the archaeological excavations that strongly influenced the design of retrofit.

The retrofit design involves the vaulted structures of the colonnade at the ground floor, the wooden elements of the gallery at first floor and of the roof, masonry walls and so on. Due to modified architectural aspects, several new steel structures (including libraries, horizontal storeys, staircases, etc.) and a global action on foundations, due to the need to bear additional loads without altering the behaviour of the whole aggregate were also designed.

The interventions hereafter described are only some of the whole ones designed and executed in the Palace, with reference to the analysis of US presented in the previous paragraphs.

US2. Open gallery at ground floor and colonnade at the first floor

The structural analysis of US2 evidenced problems in correspondence of the vaulted surfaces of the ground floor and of the wooden elements of the corresponding colonnade at the first floor. The differential subsidence due to the heterogeneity of foundations, together with the degradation of steel chains of vaults (with diffused corrosion phenomena and section's reduction) and with the presence of untied filling material led to the spreading of the cracking scenario, evidenced by in situ investigations.

As a general remark, no retrofit of foundation system has been designed basing on the results of the continuous monitoring of the actual subsidence. The execution of a global retrofit of foundations can, in fact, lead to a global modification of the current structural behaviour of the historical building, difficult to be predicted and with possible unexpected consequences on the aggregate. The installation of a continuous topographic monitoring system, able to control the development of the subsidence phenomena, has been then rather preferred, allowing to have the possibility to immediately operate in case of critical modifications of the current condition.

The retrofit of the vaulted surfaces consists in the removal of the filling material on the extrados of the vaulted surfaces—with the following decrease of the dead load—and in the restoration of cracks using specific materials for historical masonry buildings (Fig. 20a); a similar retrofit procedure is also suggested for cracks present at the intrados of the vaults. The restoration was executed following, beside the indications of actual technical standards for constructions, the prescriptions required by the Superintendence of Architectural and Historical Heritage. For example, the use of pozzolan mortar instead of the common cement mortar and the employment of wooden elements instead of steel profiles were promoted to preserve the original configuration and nature of the historical building. The introduction of thin slab of pozzolan mortar (about 3.0 cm of thickness with steel mesh of diameter 3.0 mm) and the retrofit of existing steel chains (by removal, substitution or protection against corrosion through passive systems—able to avoid the presence of non forced thrusts) provide an improvement of the structural behaviour without altering the structural feature of the aggregate.

Some problems are present, moreover, in correspondence of the first floor colonnade, with relative displacement between the wooden secondary elements of the roof and the masonry wall. The problems depend on a low support length equal to few centimeters and to the effects of daily thermal variations. In order to prevent the pull out of such elements, an additional steel profile shall be placed in correspondence of the whole perimeter of the

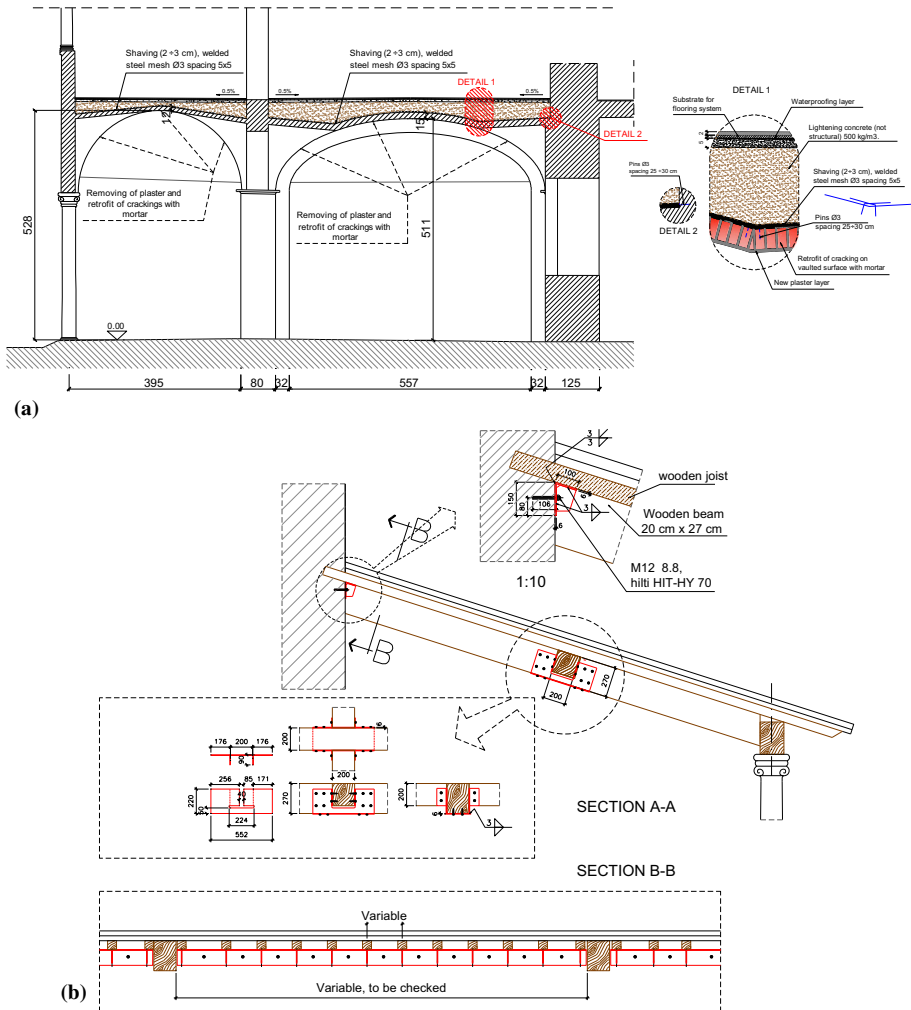


Fig. 20 **a** Intervention foreseen for the retrofit of the vaulted surfaces of the ground floor arcade; **b** retrofit intervention foreseen to provide adequate support to the wooden elements of the first floor colonnade (US2)

colonnade (Fig. 20b), providing a final support length of about 10 cm. Also in this case, the retrofit allows to reach a global behaviour of the colonnade without directly interact with adjacent masonry walls.

US3. Aula Magna and adjacent rooms at the ground floor

To solve the problem of the masonry infill wall directly bearing on the vaulted surface close to the Aula Magna at the ground floor that cannot be demolished due to the presence of horizontal storeys directly loading on it, the introduction of a steel HEA profile was accurately designed.

The introduced profile is able to sustain the masonry wall and the rest of the superstructure, including the vault at first floor and the corresponding masonry wall at the second floor. The design of the bearing profile takes into account both strength and stiffness

aspects—these last ones to avoid the vertical deflection of the masonry wall—and the study and organization of the assembly procedure (Fig. 21b). A temporary bearing structure made up of two twin truss beams with transversal support steel beams with spacing equal to 50–70 cm shall be used, allowing safety conditions during the phases of cutting of the masonry wall and introduction of the HEA bearing beam.

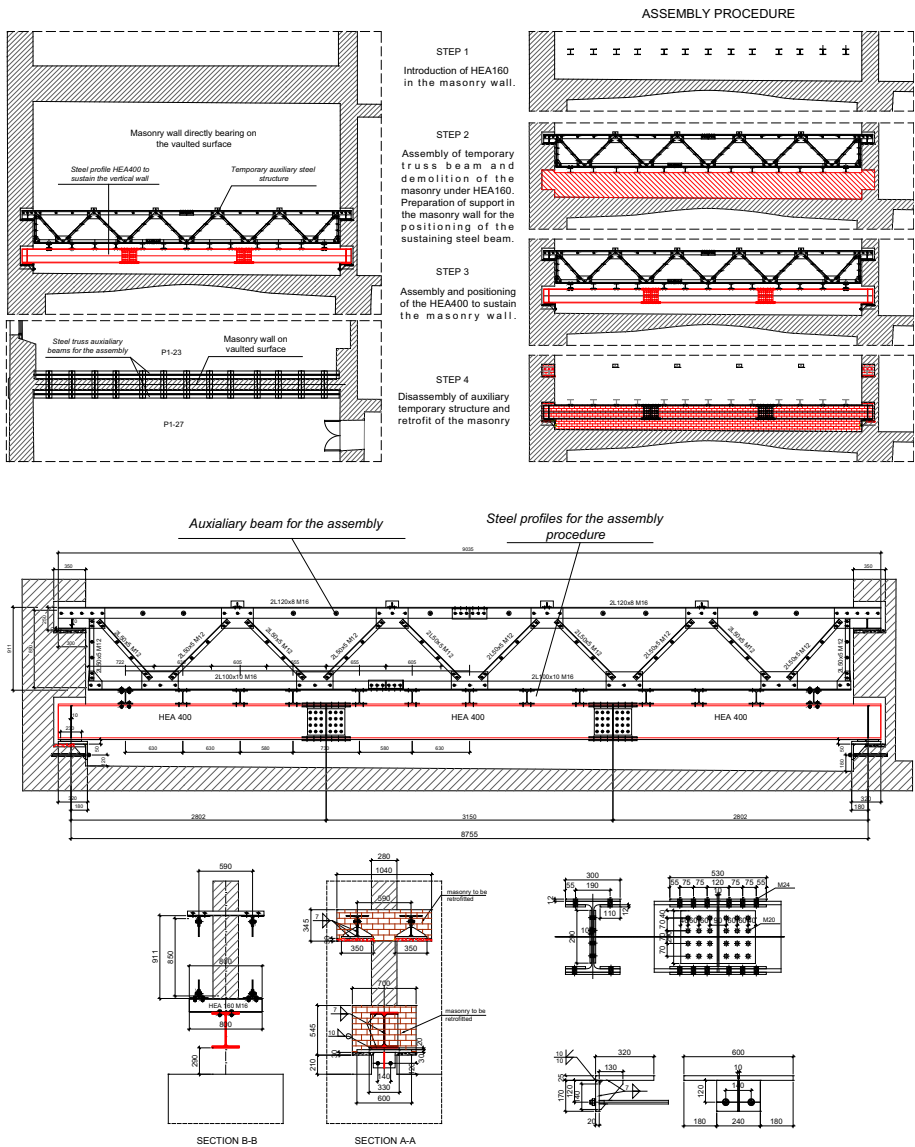


Fig. 21 Introduction of steel profile and assembly procedure to avoid the load directly upon the vaulted surface close to the Historical Aula Magna (US3)

US4. Double volume of the Aula Magna Nuova at the first/second floor

The analysis of US4 evidenced, as already mentioned, the presence of brittle brick elements constituting the pseudo-vaulted structure of the double volume of the Aula Magna Nuova, made up of reinforced-brick trusses with lightening elements of reduced thickness. The support length of such elements is about 2–3 cm, with following risk of possible cracking and collapse. In order to increase the safety margin without altering the

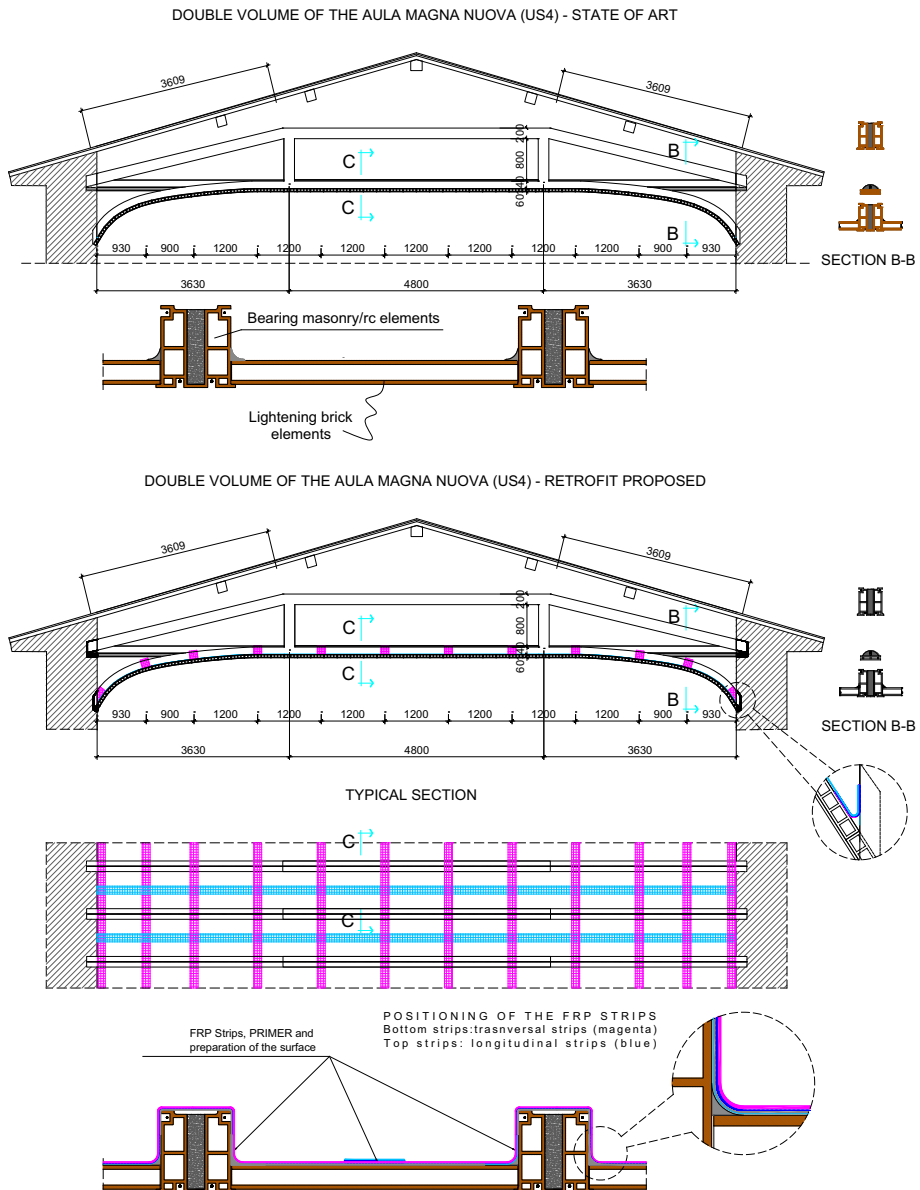


Fig. 22 Scheme of the retrofit foreseen for the vaulted surface of the Aula Magna Nuova

original structural configuration of the roof system, the retrofit proposed in Fig. 22 was designed.

The retrofit consists in the introduction of FRP strips in the two main directions, allowing joining the brick elements in the longitudinal direction avoiding the possible failure (i.e. strips with blue color) and providing, at the same time, the connection between the lightening brick elements and the trusses (i.e. strips with magenta color). The described operations do not alter the behaviour of the roof system, since no additional restraints between masonry walls and horizontal storeys are introduced. At the same time, the retrofit allows to achieve a sufficient increase of the structural safety to the building and to users.

US7. Second floor of the palace and roof

In correspondence of the storeys of the second floor of the building, some infill panels, directly loading on lightening brick elements, are present. Taking into account the need due to the global enhancement of the Palace from the architectural/functional point of view, the demolition of such panels was foreseen.

In situ investigations highlighted the presence of steel IPN profiles and brick curved lightening elements positioned at a distance equal to about 90–100 cm; the infill panels, on the other hand, in some cases directly replaces the steel profile in the sustain of the horizontal storey. In such cases, before the demolition of the infill panel, the organization of the retrofit presented in Fig. 23 has been required. Two additional steel profiles type UPN shall be introduced just upon the lightening elements, allowing beading both the storey slab and loads acting on it avoiding problems of failure during the demolition operations.

Roof structure, as mentioned in the previous paragraphs, was subjected to safety checks according to actual standards' prescriptions, based on technological investigations aiming to define the maintenance state and the level of damage due to insects, humidity and so on. Several criticisms are present in correspondence of the wooden primary and secondary elements, and, moreover, on the connections between elements (both with steel and wood structure). The situation surveyed revealed the need of diffuse retrofit interventions, that shall be evaluated time by time and connection by connection, stating the heterogeneity of solutions due to interventions executed during the years. In some cases, for example, it was necessary to replace both primary and secondary elements; in some other situations, the retrofit was limited to the wooden joists.

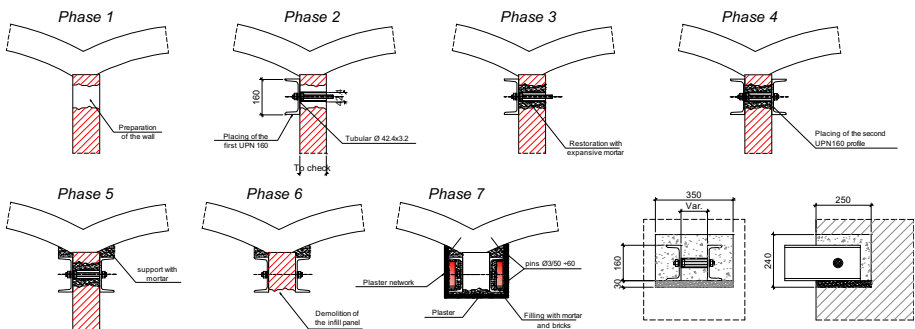


Fig. 23 Scheme of the retrofit foreseen for the demolition of the infill panel of the second floor

8 Conclusions

In the present paper, a new Knowledge-Based-Approach (KBA) for the analysis of the static safety and seismic vulnerability assessment, with following design of retrofit interventions, of an historical masonry building is proposed and applied to the monumental construction of Palazzo La Sapienza in Pisa.

The multi-disciplinary knowledge of the main features of the building, including architectural, structural, geotechnical, geological and material aspects, together with the deep investigation of the morphological evolution of the same allows to determine, in the main context of the aggregate, the different structural units/portions constituting the construction. The whole behaviour of the aggregate is then fully understood only if the single structural units are well defined in their mutual interrelationships and interactions. The disaggregation of the structural aggregate into the main US allows the critical analysis of the building without the strong computational effort related to the adoption of complex and refined numerical nonlinear models, often not able to fully represent the high heterogeneity of the Palace.

The proposed approach is based on five main steps, allowing to define the main retrofit interventions as natural prosecution of the critical analysis of the behaviour of single structural units/portions, especially taking into consideration that the retrofit of an historical building cannot be designed without preserving the original nature and configuration of the cultural heritage.

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