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Vulnerability and seismic improvement of architectural heritage: the case of Palazzo Murena

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Abstract. The aim of the present contribution is to consider and underline the essential interactions among the historical knowledge, the seismic vulnerability assessment, the investigation experimental tools, the preservation of the architectural quality and the strengthening design in regard to architectural heritage conservation. These topics are argued in relation to Palazzo Murena in Perugia, designed in the eighteenth century by the famous Architect Luigi Vanvitelli, and currently headquarters of the city's University. Based on the surveys and the visual inspections, a preliminary a priori global analysis has been performed by means of the FME method. The obtained results permitted to plan an experimental tests campaign inclusive of structural health monitoring. The new achieved "knowledge" of the building allowed to refine the seismic safety assessment. In particular it was highlighted that the "mezzanine floor" can be a vulnerable element of the building with the collapse of its masonry walls. Preserving the architectural characteristics, a local reinforcement intervention is proposed for the above-mentioned level; this consists of the application of plaster with FRCM, assuring an adequate strength, without burden the masonry structure with additional weight, and therefore a decreasing of the seismic vulnerability. The necessity to consider, in this ongoing research, other local mechanisms is highlighted in the unfolding of the last part of work.

Keywords: architectural heritage preservation; irregular masonry structures; pushover analysis; damage assessment, F.R.C.M strengthening

1. Introduction

The seismic vulnerability assessment of historic buildings has aroused, in recent years, a growing interest in the context of cultural heritage preservation. The importance of such an evaluation is obvious planning consolidation interventions, which have both to ensure both greater safety to seismic action and to conserve the historical and cultural identity of iconic buildings. Moreover historical structures, by their proper nature, have high complexity, because limited information on plans and materials are usually available and the architectures, in their current configuration, are often the result of numerous and subsequent interventions carried out over the centuries. In the perspective of a seismic analysis, there are several topics to deal with: geometrical complexity of the architecture, evaluation of the parameters regarding the behaviour of the system, definition of the scenarios that should be considered and methods of risk assessment; matters wisely studied by numerous authors (Berto *et al*. 2017, Betti *et al*. 2010, Betti *et al*. 2011, Castori *et al*. 2017, Cattari *et al*. 2014, Castellazzi *et al*. 2017, Cavalagli and Gusella 2015a, Cavalagli and Gusella 2015b, Clementi *et al*. 2016, Degli Abbati *et al*. 2019, Formisano and Marzo 2017, Fortunato *et al*. 2017, Lourenço *et al*. 2013, Ponte *et al*. 2019, Šejnoha *et al*. 2008, Valente and Milani 2019). These aspects have been recently introduced in building codes, with specific reference to the analysis of existing structures. The approach proposed, since 2008, by the Italian Building Code (NTC 2018) and argued in the recently released decree issued by the competent Ministry is based on the introduction of Confidence Factors to reduce the resistance values of materials for safety reasons. These coefficients are associated with appropriate Knowledge Levels depending on the knowledge of the asset in terms of geometrical survey, analysis of construction details and material properties. According to this procedure the seismic vulnerability assessment can be carried out by non-linear static analysis in the framework of a global seismic analysis. It is noted that the last decree also pointed out the matter of the local collapse mechanisms affecting the masonry structures. Those aspects are addressed in this paper with reference to a real case study: the seismic vulnerability analysis of Palazzo Murena in Perugia, designed by the 18th century Italian architect Luigi Vanvitelli and part of the namesake building complex, Fig. 1.

The Architecture is located in the Elce's district, consists of three adjacent buildings: Palazzo Murena (former monastic complex), the former Accounting Office and the Church of Montemorcino Nuovo or Church of the University.

In this paper is examined Palazzo Murena, headquarters

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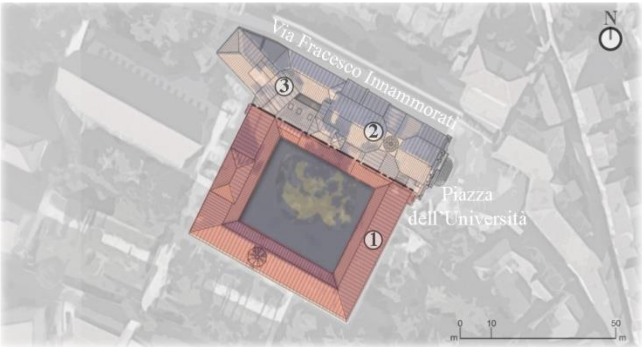


Fig. 1 Composition of the architectural complex: 1) Palazzo Murena 2) University Church 3) Ex-Accountancy

Fig. 2 Current view of Palazzo Murena

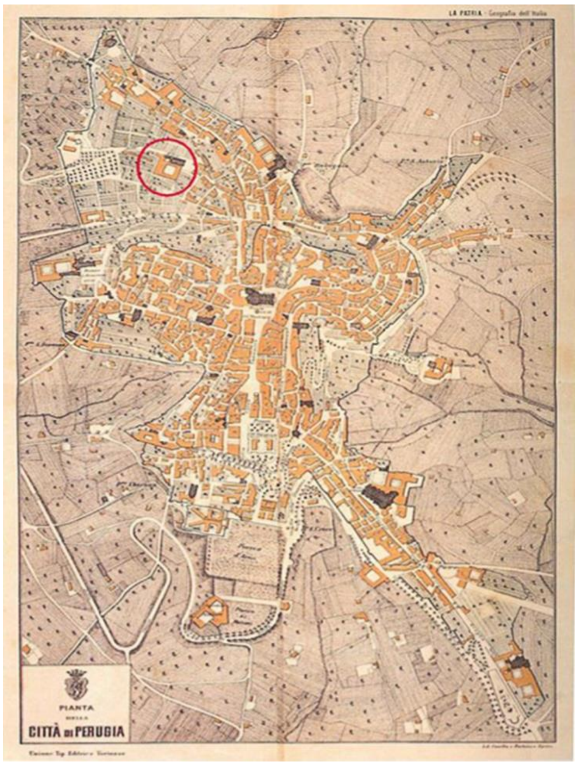


Fig. 3 Historical planimetry of the city of Perugia; the architectural complex of Palazzo Murena is highlighted by a red circle. Taken from Tip. e lit. Camilla e Bertolero (1895), La Patria-Geografia d'Italia, Unione tip. Editrice Torinese, Torino, Italy

Fig. 4 Photo of the architectural complex in the 70's

of the University of Perugia and current location of the Rector's office, Fig. 2.

2. Historical background

The creation of a the new Olivetan settlement was necessary because in 1735 the ancient seat of Monte Morcino Vecchio, located outside the Santa Susanna gate in particular above the sources of water of San Galigano, threatened to collapse due to a landslide movement. The new monastic complex, once called Monte Morcino Nuovo, was therefore arranged in the actual University Square; it was the best exposed slope of the medieval district known as the Conca of Perugia, Fig. 3. Luigi Vanvitelli designed the church of the Olivetans and the former convent of which the construction works were finalized by his pupil Carlo Murena from whom the Palace takes its name.

In fact Luigi Vanvitelli accepted the call of the Olivetans while he was working in Ancona therefore he directed the work of the church and entrusted the construction of the monastery to his student C. Murena; some design drawings are preserved in Caserta (Gianfrotta 2000). On 28th July 1740 Giorgio Cesarei, Abbot of the Olivetan Congregation, laid the first stone of the architectural complex. Palazzo Murena hosted the Olivetan monks until 1809, when the Congregation was abolished and the building was expropriated by the Napoleonic Government. In 1810, a Napoleonic law granted the use of Palazzo Murena to the University of Perugia. In 1814 however, with the return of the pontifical government, the legitimacy of the acquisition was disputed and a new act of transfer was asked by the University of Perugia to the Papal States. Despite the opposition of the religious orders, in 1815 the pope recognized the right to use of the entire complex to the University. Finally, in 1921, the municipal authority assigned the buildings definitively to the University of Perugia. From the 1970 onwards, Fig. 4, on the project of G. Nicolosi, the Faculty of Geology was established close to the Olivetan complex (Grohmann 1981); furthermore, by renovating the square in front of the ancient monastery, a monumental staircase was created to provide access to the area above. Later, during the G. Ermini direction, the number of University buildings increased, expanding over the neighbourhood and towards the old town.

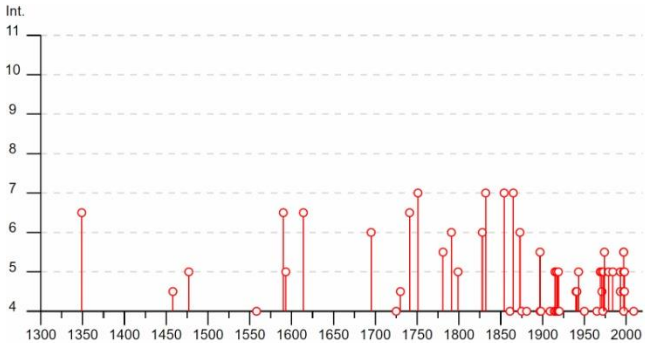
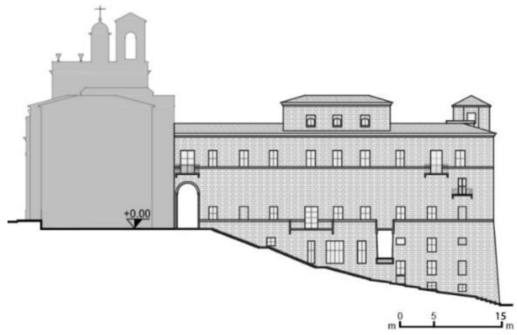
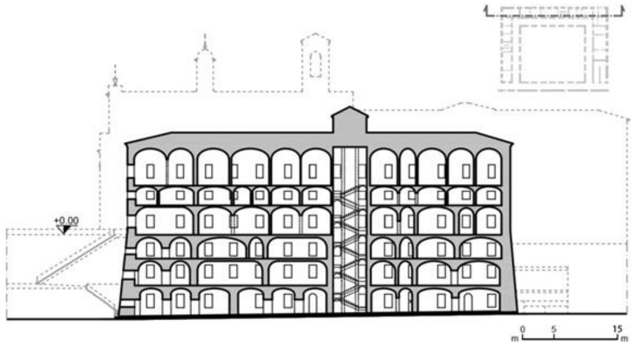


Fig. 5 Intensity of the earthquakes (Mercalli scale) of the last centuries concerning the area of Perugia. Taken from CPTI15 Parametric Catalogue of Italian Earthquakes 2015, INGV, Italy. www.ingv.it



(a) Elevation northwest



(b) Cross-section

Fig. 6 Geometrical survey

During the last centuries, a considerable number of seismic events affected the city, Fig. 5. The I.N.G.V. (National Institute of Geophysics and Volcanology) for the area of Perugia provides information about the main earthquakes occurred from the construction of Palazzo Murena: Sellano - October 11th, 1791; Cannara - February 12th, 1854; Norcia - August 22nd, 1859; Montefalco - September 15th, 1878; Cascia - February 23rd, 1879; Monterchi - April 26th, 1918; Norcia - September 19th, 1979; Gubbio - April 29th, 1984; Colfiorito - September 26th, 1997; Perugia - December 15th, 2009 (Rovida *et al*. 2015). In particular, the earthquake of 1997 caused significant damages to Palazzo Murena, which are mainly located in the northwest area of the building. After this event, the building was interested by consolidation interventions. So it appears clear that, during time, the building has been subjected to changes, also in its use, which determined the modification of the architectural-structural arrangement and the possible variation of its seismic vulnerability. At present, Palazzo Murena hosts the headquarters of the University of Perugia.

3. Actual state of the building

3.1 Geometrical surveying campaign

The Palace has been the subject to a surveying campaign, commissioned by the University of Perugia, aimed at an evaluation of its relationship with adjacent buildings and a better understanding of the architectural distribution, Fig. 6. Palazzo Murena is built on a hillside in the northwestern periphery of the historic centre of Perugia, in a neighborhood of medieval origin, at *440* m above the sea level. The urban layout of this portion of the historic hill of Perugia is arranged with a series of terraces, that host hanging gardens, stabilized with retaining walls, in general, from *4* to *8* m high. The average slope is around *27-28 %*. The building is divided into six floors: three floors are completely out of ground and the other three floors are half-buried, since they have free sides facing the valley of the Conca. The former monastery is an imposing quadrangular structure with a central courtyard, Fig. 7.

Its brick frames facades are austere, splitted by the regular openings and articulated by the string-courses corresponding to the floors; also the shape of the three basement levels, facing the valley, contribute to its sturdy appearance. The ground floor has rectangular shape, whose dimensions are about 60 m x 48 m, with an inner cloister, having sides about 30 m x 33 m. The heights of the levels are: third basement - 4.80 m, second basement - 4.10 m, first basement - 3.80 m, ground floor - 4.75 m, mezzanine floor - 3.10 m (Fig. 8), first floor - 6.00 m. Three types of vaults can be observed: cross vault, barrel vaults and barrel vaults with cloister heads The indoor hallway is adorned by plasters and mouldings and there are also rooms decorated by fresco like the “PhD’Room” at the last floor, Fig. 9.

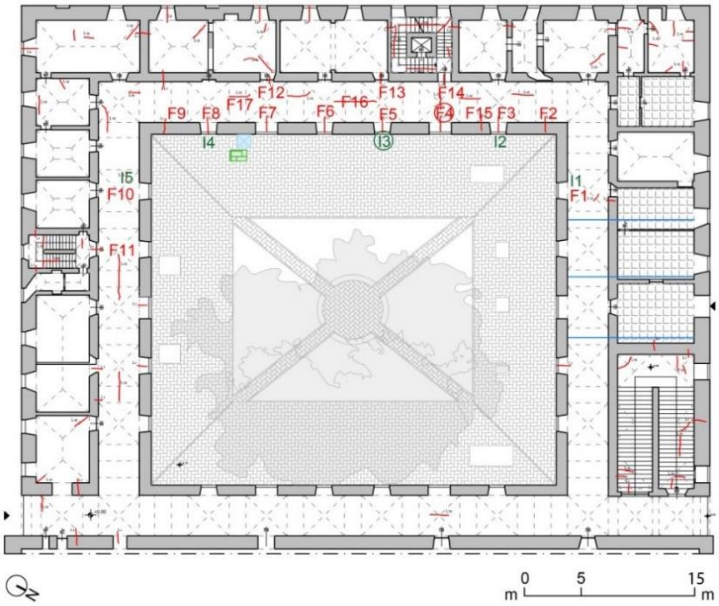
3.2 Structural and damage survey

In order to outline a preparatory analysis of the seismic vulnerability, visual and localized investigations were fulfilled at the same time of the survey; these consisting of:

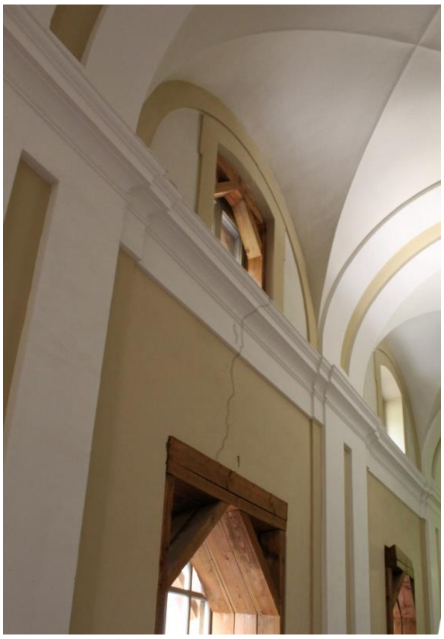
- inspections of the masonry walls conducted, after the removal of the plaster, in correspondence of areas devoid of frescoes or elements with significant architectural value;
- monitoring of the cracking pattern through inclinometers and thermo-hygrometer.

The palace is almost exclusively built of stone or brick walls and brick vaults and the visual inspections allowed to identify two main types of walls belonging to the categories proposed by the Italian Building Code (NTC 2018):

- splitted stone masonry with good texture;
- masonry in solid bricks and lime mortar.



(a) Plan with the cracking pattern (red lines) and location of the monitoring instruments: F1-F17 crackmeters, I1-I5 inclinometers; in pale blue and green the inspections on mortar and masonry respectively; in blue lines the tie rods installed after 1997 earthquake



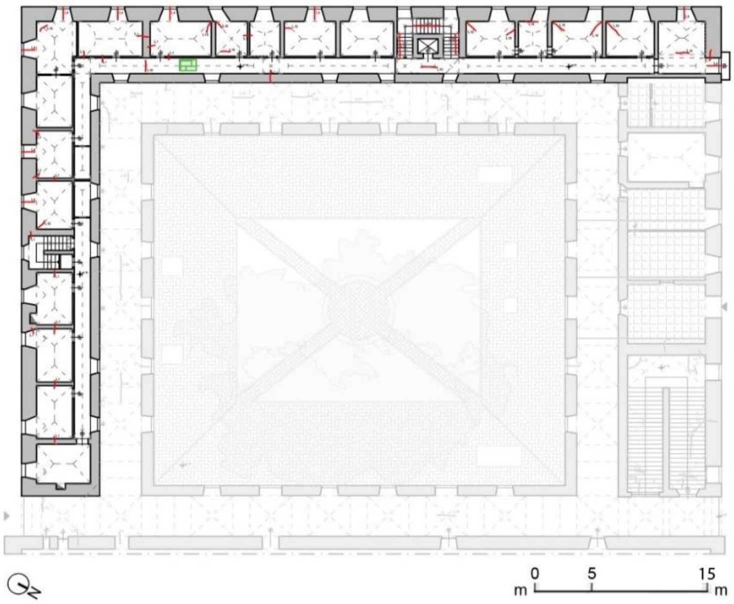
(b) Side of the main corridor characterized by cracks on the masonry architraves and vaults due to the 2009 earthquake

Fig. 7 Ground floor

The building consists of three above-ground levels and three partially basement floors; it follows a great variability in the dimensions of the load-bearing walls. The wide range of thicknesses of the perimeter walls is between 2.20 m, starting from the lower storeys where masonry buttresses are observed, and about 0.6 m for the masonries of the top floor. Concerning the cracking pattern, local phenomena have been highlighted, in particular damages on architraves and cracks at the intrados of the vaults and the arches, Fig. 7, 8 and 9. The interpretation of the cracking pattern also enabled to deduce the possible local mechanisms that could be activated by the seismic action (Adem and Halil 2012).

The most significant structural deficiencies observed, are

• thrust exerted by the gabled roof on walls;
• walls of the mezzanine floor, not vertically continuous to the foundations and therefore
• masonry partitions with small thickness which have bearing function;
• cracks on walls along one side of the corridor on the ground floor adjacent to the cloister;
• cracks at the intrados of the vaults on the ground floor;
• deterioration and localized cracks in the wooden architraves;



(a) Plan of the mezzanine floor; in red is characterized the cracking pattern and in green is reported the location of the visual investigation on a wall declaring the presence of masonry in solid bricks and lime mortar

(b) Side corridor to the ancient monks' cells now used as offices

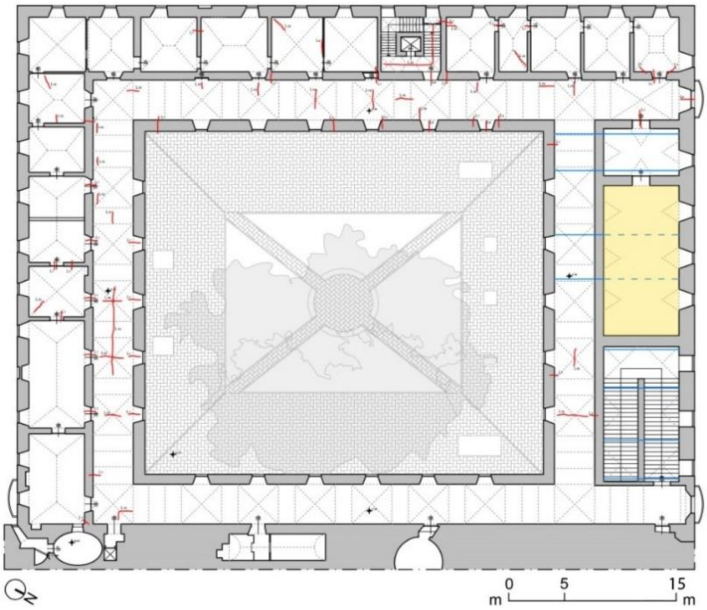
Fig. 8 Mezzanine floor

- widespread deterioration of the masonry walls with lack of mortar between joints;
- localized situations of high degradation and damage (e.g., on the roof);
- portions of floors characterized by different plane stiffness also due to the partial consolidation performed after the 1997 earthquake.

4. Preliminary seismic global analysis

4.1 Introduction

The experimental tests and investigations, especially the partial destructive and the destructive ones, are expensive and demanding. Moreover they inevitably lead to partial damages of the architectural artefact. With the aim of restricting the execution of this type of tests only on the most significant areas of the building, a preliminary modelling of structure was carried out. The results of the geometric survey together with the visual investigations highlighted the main structural elements, their dimensional connotations and, consequently, their mechanical characteristics attributable a priori from those provided by the NTC (2018). The outcomes of this first analysis allowed to identify the vulnerable portions of the masonry structure and most critical points for each floor, confirming some of the structural deficiencies identified during the survey



(a) Plan of the first floor; in red is characterized the cracking pattern, in blue lines the tie rods installed after 1997 earthquake; the region in yellow highlights the "PhD'Room"

(b) Detail of the fresco in the "PhD'Room"

Fig. 9 First floor

Table 1 Range of mechanical parameters extracted from compression strength (f_m); shear strength in absence of normal stresses (τ_0); modulus of elasticity (E); tangential elasticity modulus (G); specific weight (w)

Wall type		f_m [N/cm²]	τ_0 [N/cm²]	E [N/mm²]	G [N/mm²]	w [kN/m³]
MS*	min	260	5,6	1500	500	21
	max	380	7,4	1980	660	
MB**	min	240	6	1200	400	18
	max	400	9,2	1800	600	

campaigns, on which the subsequent experimental tests were focused (Seker *et al*. 2012).

4.2 Types of walls and mechanical characteristics

We recall that various investigations held during the survey permitted to locate the main recurring types of walls. The prescribes the reference values of the mechanical parameters and the specific weight for some types of recurring walls on the Italian territory; in the Table 1 are shown the variability intervals relating to the types of walls identified within Palazzo Murena.

4.3 Knowledge levels and confidence factors

The Italian Building Code (NTC 2018) defines three increasing Knowledge Levels: LC1, LC2 and LC3. The definition of the knowledge level for the building depends on three aspects, i.e., knowledge of the geometry, of the construction details and of the material properties. Considering, in this phase, the lack of experimental tests, for both types of the materials cited at *§* 4.2 a knowledge level LC1 was assigned, corresponding to a "limited knowledge" of the building. When the knowledge level LC1 is assigned, a value of the Confidence Factor (FC) equal to 1.35, the most penalizing, is prescribed for both types of walls. In non-linear static analyses, mechanical parameters of the masonry must be divided by the Confidence Factor to obtain design values

$$f_d = \frac{f_m}{FC} \tag{1}$$

being f_m, in LC1, the minimum values of the intervals reported in Table 1 and f_d its design value; for the stiffness hence must be used the average values from the intervals above reported. Furthermore, the flexural and shear stiffness of the masonry elements was reduced by 50% to take into account the presence of cracks. The final design values of the mechanical parameters are shown in Table 2.

4.4.1 Masonry vaults

The studies conducted have also showed the presence of four main types of brick vaults: cloister vaults, cross vaults, barrel vaults and barrel vaults with cloister head. It should be noted that Palazzo Murena has numerous architectural peculiarities such as rather articulated load bearing structure and floor to floor variable plans in relation to the elevate e.g. the mezzanine floor. For these reasons the vaults have been modelled as equivalent deformable slabs, different for each vault's typology. This modelling approach is the result of specific studies conducted in order to create an equivalent diaphragms model for the vaults starting from the F.E.M. analysis of such masonry elements (Cattari *et al.* 2008, Lagomarsino *et al.* 2013). Regarding the permanent loads, in this first phase, were used the respective default values for each type of masonry vault (Giresini *et al*. 2017).

4.5 Macro-element modelling

The structural analysis of the building is performed by means of the software 3Muri (STA DATA 2018), which makes use of the *F.M.E.* method (Frame by Macro-Element). Following this approach (Lagomarsino *et al.* 2007), the in-plane behaviour of the masonry wall is modelled through an equivalent frame: each wall of the building is subdivided into piers and spandrels (2 nodes macro-elements) connected by rigid areas (nodes). The non-linear macro-element model, originally proposed by Gambarotta *et al.* (1997), permits to reproduce the two main in-plane failure modes of the masonry panels, i.e. bending-rocking and shear-sliding mechanisms (with friction), with a limited number of degrees of freedom. The bearing structure under vertical and horizontal loads is identified with walls and floors (or vaults): the walls are the bearing elements, while the role of floors consists in sharing vertical loads to the walls and generating a planar stiffening effect.

Table 2 Design parameters related to the walls identified for the non-linear analysis

Wall type	FC	Reduction factor	f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
MS	1,35	50%	193	4,2	870	290	21
MB	1,35	50%	177	4,4	750	250	18

In this first phase, exclusively the seismic global behaviour of the structure is studied; in a future perspective the authors envisage the analysis of the possible local mechanisms concerning Palazzo Murena. Since the seismic global building response is governed by the in-plane behaviour of bearing masonries, it follows that the local flexural behaviour of the floors and the walls out-of-plane failure is neglected (Lagomarsino *et al.* 2007).

4.6 Non-linear static analysis

The seismic vulnerability assessment is carried out by the pushover analysis. Such non-linear static analysis permits to evaluate the displacement capacity of the structure, corresponding to a maximum value of acceleration that the structure can withstand before reaching the ultimate limit state. The methodology consists in applying to the structure the gravitational loads and a system of horizontal forces at each level of the building, as specified in the following, and having resultant force V_b. The horizontal forces are proportionally increased, until the construction reaches the collapse, so that the displacement d_c of a "control node" increases in a monotonic trend. At each load step, for each masonry panel, the maximum drift prescribed by the code is checked, corresponding to 0.4% and 0.6% of h respectively for shear and bending collapse, being h the height of the panel. If such thresholds are exceeded, the panel is no longer considered to be able to withstand the horizontal actions and it is replaced by a strut, which can share normal forces but cannot carry seismic actions. The analysis stops when the model does no longer support additional horizontal load increments (Pantò *et al.* 2018). The results of the non-linear static analysis may be summarized in a Cartesian plane, having in abscissa the displacement d_c of the control node and in the ordinate the total shear force V_b. The obtained pushover curve d_c-V_b identifies the lateral response of the building. As known, an equivalent *Single Degree of Freedom* (SDOF) system is associated to the masonry structure so that an equivalent bi-linear pushover curve shall be determined. The forces V^* and d^* of the equivalent SDOF system are obtained respectively from V_b and d_c, dividing them by the *modal participation factor* (Γ) deriving from the formula C7.3.5 that is $(\sum m_i \phi_i \tau i)/(\sum m_i \phi_i^2)$, where m_i is the *ith* mass and Φ_i is the *ith* component of the displacement of the first mode normalized to the displacement of the control point for any applied system of forces and τ_i is the *ith* component of the displacement corresponding to unitary displacement in seismic direction (Circ. 7/2019). Being V_{bu} the maximum force of the real pushover curve, V_{bu}^* the corresponding value for the equivalent SDOF system is equal to V_{bu} / Γ. The pushover curve of the equivalent

SDOF system is replaced by a bi-linear curve, having a first elastic side and a second plastic side. The bi-linear pushover curve is determined by imposing that its elastic side passes for the point $0.7V_{bu}$ of the pushover curve of the equivalent SDOF system; the yielding force V_y^* is obtained by an equivalence between the area subtended by the pushover curve of the equivalent SDOF system and that subtended by the bi-linear curve, for the displacement d_c^*, which corresponds to a reduction less than or equal to $0.2\ V_{bu}^*$. Hence, the elastic period of the bi-linear curve is

$$T^* = 2\pi\sqrt{\frac{m*}{k*}} \tag{2}$$

with m^* calculable from the formula C7.3.6 that is $\sum m_i\phi_i\tau i$ and $k^*$the stiffness of the elastic side of the bi-linear curve. The pushover analysis is conducted in terms of displacement, it is carried out by comparing displacement capacity d_u^* and demand d_{max}^*. The structure's safety is verified if d_{max}^* is minor than d_u^*. The NTC (2018) provides further details for the evaluation of the displacement demand d_{max}^*, depending on the elastic period of the bi-linear system. Being q^* the ratio between the total shear at the base of the equivalent SDOF system determined by means of the elastic spectra and the total shear obtained by the non-linear static analysis it must be $q^* \leq 3$. As mentioned the global seismic analysis involves the entire construction and engages the walls in their plan. The mechanisms concern single piers or limited portions of the masonry structure and derive from the lack of connections between walls and slabs; the pushover analysis carries on, despite the crisis of the first elements, as long as the piers of a same floor do not break simultaneously. Again, in relation to the geometrical-architectural complexity of Palazzo Murena, sensitivity analyzes (Pagnini *et al.* 2011, Rota *et al.* 2014 and Tondelli *et al.* 2012) were conducted in order to achieve the best modeling strategy for the mezzanine floor and to locate the reference node for the purpose of reach a satisfying representation of the structure's actual seismic behaviour, Fig. 10. Furthermore, according to the NTC (2018), the results of the pushover analysis can be expressed in terms of the coefficient α_{SLX}, defined as

$$\alpha_{SLX} = \frac{PGA_{CLX}}{PGA_{DLX}} \tag{3}$$

where PGA_{CLX} is the limit capacity acceleration, or the maximum entity of the actions, considered in the planned design combinations, that the structure is able to support and PGA_{DLX} is the spectral demand acceleration for each limit state, or the reference value of the acceleration of the seismic action. X is indicating the considered limit state: C for the collapse limit state, V for the life-saving limit state, D for the damage limit state and O for the operational limit state. In particular, a value of α_{SLX} equal or greater than *1* indicates the fulfilment of the simulation. As prescribed by the NTC (2018), regardless of the mass participation, two distributions of horizontal forces must be considered. In this paper, the first one is chosen to be proportional to the distribution of forces obtained by static linear analysis and the second one corresponds to a uniform distribution of forces, derived from a uniform distribution of accelerations. The seismic capacity of the building is assumed to be the minimum one obtained from such analyses and in Table 3 are reported only the lowest ones for each earthquake's direction. Considering the presence of some values of the α_{SLV} lower than 0.6, the building can be counted among those at seismic risk and would require structural strengthening.

Table 3 Outcomes of the pushover analysis for the model at LC1

Dir.*	Seismic Load distribution	Ecc. [cm]	α_{SLC}	α_{SLV}	α_{SLD}	α_{SLO}
-X	Static Forces	-234.6	0.712	0.639	1.779	1.684
-Y	Uniform	296.5	0.555	0.498	1.214	1.502

* Out of a total of 24 different types of pushover analysis conducted for each model, here are reported only the heaviest ones for each seismic direction

The punctual evaluation of these results in terms of deformation and failure mechanisms of the masonry panels (Pardalopoulos *et al.* 2016) confirmed some structural weaknesses detected during the survey and also highlighted the most vulnerable floors to which the performance of the experimental trials was subsequently assigned.

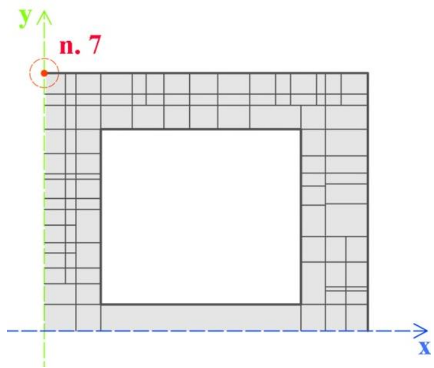
5. Seismic global vulnerability assessment

Previous results were influenced by the assumption of LC1 and consequently of a FC equal to 1.35. In order to increase the knowledge of the previously identified structural peculiarities of the building, instrumental investigations were conducted. The numerous experimental investigations carried out on the structure have allowed to reach a better characterization of the different masonry types in terms of mechanical properties. In addition the videoendoscopies, allowed to evaluate with greater accuracy the thicknesses of the load-bearing elements, and the constructive features, both for the walls and for the masonry vaults.

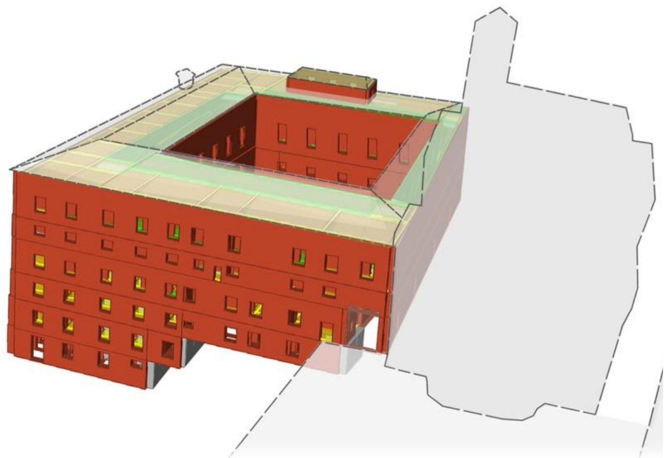
Such outcomes have been used to obtain an up-to date model aimed at the seismic assessment of Palazzo Murena.

5.1 Non-destructive and partial-destructive experimental tests

In 2016 geophysical investigations in correspondence of the inner courtyard of Palazzo Murena with execution of radar scans and electromagnetic reliefs have been carried out. Furthermore, in 2017 were conducted series of endoscopic tests in order to identify geometric and material characteristics of the vaults of Palazzo Murena. In addition, a coring in the foundation and the related video endoscopy at the inner courtyard was performed. In the last years of monitoring the actual behaviour over time of the cracking pattern has been checked as a useful tool to assess the safety of architectural heritage (e.g., Bartoli *et al.* 1996, Cavalagli *et al*. 2018, Gioffrè *et al.* 2008).

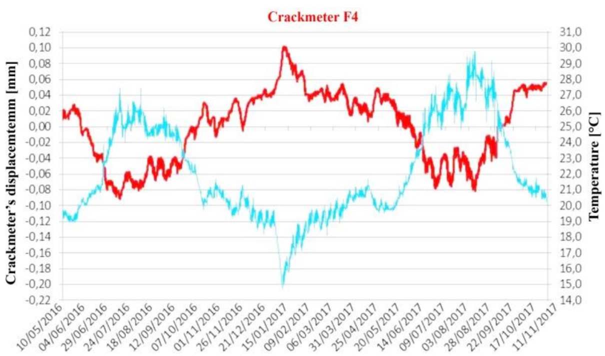


(a) Plan of the numerical model, the interior walls relative to different floors are overlapping, n.7 is the "reference node" of the roof-floor used in the pushover analysis



(b) 3D view of the numerical model

Fig. 10 Numerical model (F.M.E.) of Palazzo Murena



(a) The recordings of the crackmeter F4 are marked in red in relation to that of the thermo-hygrometer in pale blue: displacement (-) stands as crack's closure; displacement (+) coincide to an opening of the cracks

(b) In green the recordings of the inclinometer I4 are highlighted in relation to that of the thermo-hygrometer: rotation (+) correspond to a rollover of the wall towards the outside of the building, rotation (-) represent wall's rollover towards the interior of the building

Fig. 11 Some results of the monitoring systems (the location of the instruments is shown if Fig. 7)

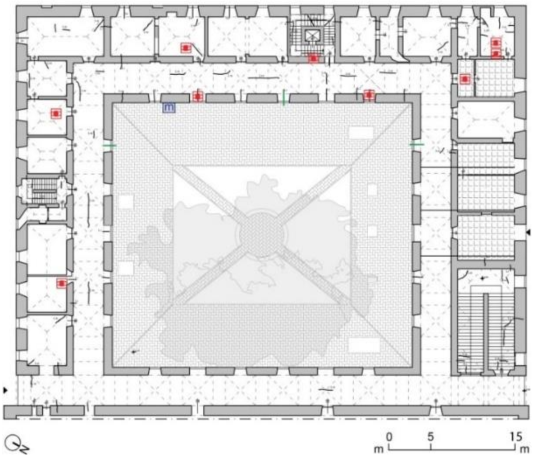
So that a monitoring system was implemented in Palazzo Murena, on 10 May 2016 and its activity was conducted until 11 November 2017.

In particular, crackmeters, inclinometers and a thermo-hygrometer sensors were used. A periodic behaviour of the lesions was observed, limited to the analysed period, in counter-phase with the thermal variations and moreover no trends were observed, Fig. 11. It can therefore be concluded that, at present day, the cracking pattern appears to be stabilized and that there are no arising deformation processes in place for the masonry structures and / or the foundations.

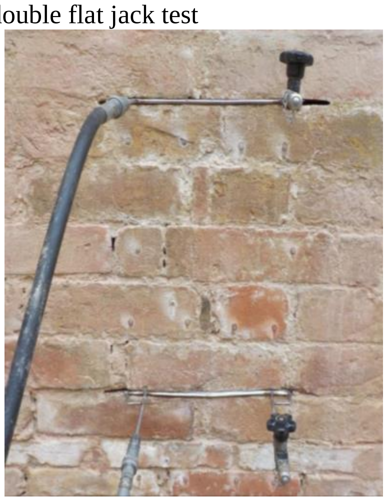
In order to evaluate the type and quality of masonry and to investigate the structures of walls and vaults, the following experimental tests, Fig. 12, have been performed:

- videoendoscopic investigations on load-bearing walls and vaults;
- single and double flat-jack tests;
- sonic testing;
- compression test on mortar samples;
- geophysical, radar and electromagnetic tests at the courtyard;
- monitoring of the cracking pattern, inclinometers and thermo-hygrometer.

The video endoscopic investigations, together with the tests on mortar samples and the tests with flat jack, have allowed to confirm the identified masonry types and to rate the quality of the walls inside the building, belonging to the already mentioned categories proposed by the Italian building code. Single and double flat jack tests were carried out on both types of masonry, permitting to evaluate the stress state, the compressive strength and the elastic modulus of the different types of masonry. The video endoscopic investigations permitted also to identify the real thickness of the examined walls; in a non-structural "false wall" and inner cavity before the bearing part of the masonry panels.



(a) Position of the experimental tests conducted on the ground floor: in red the video endoscopy from intrados, in green the performed drills, in blue the location of the performed double flat jack test



(b) Execution of the MP 2 compression test with flat jacks in order to establish the maximum compression strength (fm) and the Young's module (E) of the brick wall

Fig. 12 Some of the experimental tests conducted to increase the level of knowledge

The stone walls are mainly located against the ground, while the remaining walls of the building consist of brick and lime mortar. It is observed that most of the counter-earth walls are characterized, proceeding from the inside towards the outside, by a counter-wall in hollow brick and a cavity of modest thickness that precede the bearing wall.

For what concerns the vaults the stratigraphy shows, from the intrados to the extrados, the presence of plaster, brick, with thickness in the range of *7-29 cm*, infill, screed and flooring.

5.2 Knowledge levels and confidence factors

On the basis of the performed tests and inspections cited at paragraph § 5.1 and conducted in order to increase the knowledge of the building, a level LC2 is assigned, corresponding to an "adequate knowledge'' of the building. Rather than using the mechanical properties deriving from the single and double flat jack tests (Fig. 12), it was

Table 4 Design parameters related to the walls identified for the non-linear analysis

Wall type	FC	Reduction factor	f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
MS	1,2	50%	267	5,4	870	290	21
MB	1,2	50%	267	6,3	750	250	18

Table 5 Analysis parameters

Dir.	T* [s]	m* [kg]	w [daN]	M [kg]	m*/M [%]	Γ	F^*_y [daN]	d^*_{max} [cm]	d^*_u [cm]
-X	0,88	21x10^6	32x10^6	33x10^6	64,89	0,86	39x10^5	3,54	8,94
-Y	0,49	23x10^6	32x10^6	33x10^6	71,69	0,79	40x10^5	1,03	2,83

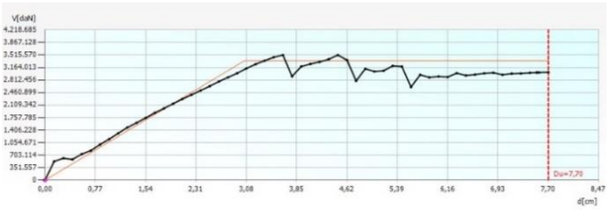
preferred to make a choice on the safe side by adopting the much less performing values of the aforementioned ones provided by the NTC (2018) according to the additional Palazzo Murena's characteristics discovered during the experimental campaign. So taking into account the results of tests to confirm the identified masonry types and in accordance with the last decree the two occurences of (Circ. No 7 2019), the mean values of the respective intervals were adopted for the resistance and for the elastic modulus of each type of wall. Hence, a value of the Confidence Factor (FC) equal to 1.2 is prescribed for both types of walls. The design values for the non-linear static analyses are showed in Table 4.

5.3 Loading system

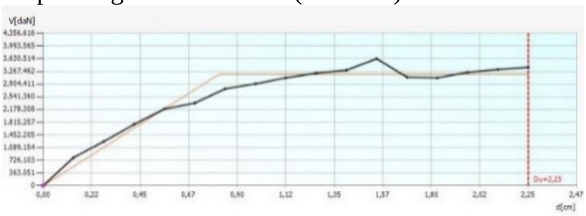
On the basis of the results provided by the video endoscopic investigations it was possible to match the different vaults to the respective permanent loads estimable in relation to the different thicknesses observed. The variable loads proved to be consistent with the previous calculation model; the same earlier evaluated seismic action has been considered.

5.4 Non-linear static analysis in LC2

In order to make the analyses comparable the model used to assess the global seismic safety of Palazzo Murena has been based on the one created for the preliminary analysis. Hence the pushover analyses were conducted in the same directions according to the previously identified reference node defining the pushover curves in order verify the safety of the structure regarding the displacements for different limit states. As mentioned, according to the Italian Building Code (NTC 2018) such outputs can be evaluated also in relation to the ratio of capacity and demand defined in terms of acceleration. In order to provide insights on the actual global seismic behaviour of Palazzo Murena are reported, for instance, the pushover curves (Fig. 13) and the respective parameters related to the aforementioned most demanding analysis. The execution of the experimental tests, in addition to having allowed the characterization of the permanent loads, offered the opportunity to pass from a LC1 to a LC2 knowledge level, with the consequent enhancement of the mechanical characteristics, of the walls, considered in the calculation.



(a) Pushover analysis conducted in direction –X for uniform seismic load distribution (black lines) and the corresponding bi-linear curve (red lines)



(b) Pushover analysis conducted in direction –Y for static forces seismic load distribution (black lines) and the corresponding bi-linear curve (red lines)

Fig. 13 Pushover curves for the model at LC2

Table 6 Outcomes of the pushover analysis for the model at LC2

Dir.*	Seismic Load distribution	Ecc. [cm]	α_{SLC}	α_{SLV}	α_{SLD}	α_{SLO}
-X	Uniform	234.6	0.997	0.896	1.498	1.854
-Y	Static Forces	296.5	0.711	0.639	1.367	1.325

* Out of a total of 24 different types of pushover analysis conducted for each model, here are reported only the heaviest ones for each seismic direction

This brought about a significant improvement in the seismic response of the building which now, still globally, not represents an immediate danger but requires structural reinforcements presenting values of the average α_{SLV} albeit slightly superior to the 0.6 threshold but still minor to 1. Also noteworthy is the remarkable enhancement of the seismic behaviour along the X direction of the earthquake, Table 6. It should be noted that the seismic vulnerability could be further reduced through an intervention of structural strengthening which must be destined to the areas of the structure of verified weakness. Analysing the results, a rank of the identified risks, which highlighted the need to conceive an improvement intervention in the mezzanine floor, was drawn up.

6. Intervention of structural strengthening

In the context of a seismic improvement intervention, the NTC (2018) establish at § 8.4.2:"The safety assessment and the improvement intervention, must be increased by a value not less than 0.1 to all the parts of the structure potentially affected by behaviour changes, as well as to the structure as a whole. For the seismic combination of actions, the value of ξ_E may be less than unity. Except for specific situations relating to cultural heritage, for class III buildings for school and class IV use, the value of ξ_E, following the improvement interventions, must in any case be no less than 0.6, while for the remaining buildings of class III and for those of class II the value of ξ_E, again following the improvement interventions, must be increased by a value not less than 0.1. In the present contribution α_{SLV} stands as ξ_E representing the ratio relationship between the building's ability to withstand the earthquake and the demand in terms of seismic action that is required of the building at that particular site. It must therefore observed, § 5 that the current state of Palazzo Murena is already characterized by a value of α_{SLV} slightly higher than 0.6. Nevertheless it would be appropriate to design a structural reinforcement plan aimed at to increase the seismic safety of the building.

Possible interventions of structural strengthening derive from the knowledge of the constructive specificities and of the building's uniqueness. The conceiving of such design actions, that as we know are never predictable or predefined, must prevent the dangerous factors related to the artifact's survival without jeopardize the role and the meaning they stood for during the centuries; on this, today new reinforcement methodologies include the use of next-gen. composite materials. These materials, that can be studied by analytical approach, with classical and non-classical theories (Autuori *et al.* 2017), numerical allow to fulfil the strengthening of ancient masonry structures in compliance with the environmental, cultural and social context where such buildings are placed. In order to envisage intervention of structural strengthening the design philosophy, here adopted, was not to burden the masonry structure with multiple and widespread interventions but rather to retrofit only the most vulnerable structural elements proposing a localized application of F.R.C.M (Fiber Reinforced Cementitious Matrix).

For these reasons the authors considered this structural strengthening with reference to the mezzanine floor in order to evaluate, later, its impact on the global seismic behaviour of the building (Lignola *et al.* 2018).

The awareness of the global fseismic vulnerability represented by the geometry irregularity of the mezzanine floor, also constituted by walls built without a direct load path to the ground, involves the consideration of the appropriateness, in future studies, of evaluating the possible benefits of this type of structural reinforcement also in the perspective of the local collapse mechanisms potentially equally dangerous for the seismic safety of Palazzo Murena.

6.1 Collaborative plaster reinforced with fiber mesh

The intervention consists in the coupling of a high performance fiber mesh and an inorganic matrix (mortar) as adhesive (Hadzima *et al.* 2018). The mortar replaces the traditional epoxy resins (used instead in F.R.P. systems) in order to make a real reinforced plaster. This can act as a structural reinforcement of masonry walls. Moreover, plasters made in this way have multiple benefits compared to that achieved with the traditional welded wire (Alecci *et al.* 2016a, Ascione *et al.* 2015, Carozzi *et al.* 2014, Carozzi *et al.* 2015, D'Ambra *et al.* 2018, Kimia 2019):

- good mechanical properties;

• reduced thickness compared to other traditional reinforced plasters;
• easy installation thanks to the lightness of the mesh and the modalities of application in comparison to other techniques (e.g., F.R.P.);
• reversibility of the intervention through the use of the inorganic mortar that is less aggressive, with respect to the masonry, than the epoxy resins used in other composite materials;
• recyclability of the natural origin nets considering the characteristics of the matrix that allows the chemical separation from it without damaging the quality of the fibers;
• weather resistance and great durability in aggressive environments;
• nonmagnetic and radiolucent property;
• good resistance in case of fire.

In the implementation of this intervention it is also possible to provide a cross connector to bond the two sides of a wall in order to positively influence the failure modes (Kimia 2019).

6.2 Design

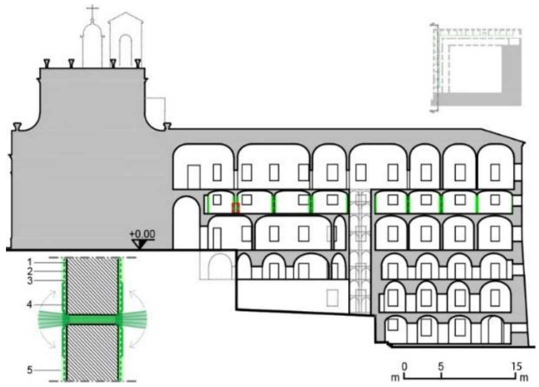
The structural concept of this improvement acknowledges the efficacy of the existing masonry structure and intend to improve its performance, in case of earthquake. The proposed design plan is aimed at the tutelage of the building's architectural value by focusing the interventions only on the elements of proved vulnerability so without involving stuccos, fine marble flooring (Fig. 8a), and other valuable elements that could not be restored if it were damaged during the building work phase. Therefore an application of F.R.C.M localized on some interior's walls, has been planned in order to strength the masonries regarding the bending and the shear actions without, besides, burden the existing masonry structures with the additional weight of structural gears non-canonical if compared to the genesis of the building.

As outlined in the preliminary global seismic analysis and subsequently confirmed by experimental investigations and by the seismic evaluation of the current state of Palazzo Murena, the level of the mezzanine floor, made of bricks and lime mortar, has been identified as a vulnerable zone of the masonry structure.

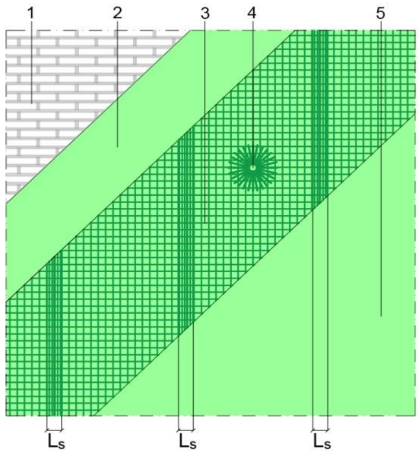
This level, in addition to represent a large geometric irregularity and to provide a huge difference in the distribution of the structural masses, consists of load bearing masonry walls not vertically continuous to the foundation and therefore resting, in turn, on the underlying brick vaults (De Santis *et al*. 2019). The design criteria and the technical checks with which this intervention was dimensioned followed the guidelines provided by the CNR (CNR-DT 215/2018) and led to a widespread application of the aforementioned "reinforced plaster" applied on both sides of the internal walls constituting the ancient cells of the monks for a height that ends at the springer quote of the overlying masonry vaults (Alecci *et al*. 2016-b, Alecci *et al*. 2017).

For design purposes only, the mechanical parameters supplied by Kimia S.p.A. (2019) were taken as reference including the use of transversal connectors known as "sfioccature", in order to create artificial through stones within the walls, and to lead to a sandwich masonry-reinforced plaster panel. In addition, using the same strategy the terminal part of the intervention was cuffed up on the corners between internal and perimetral walls, according to an anchoring length of 20 cm, in order to further constrain such masonry elements. To recap the constructive reinforcement procedure consists of 5 phases reported contextually to the design drawings, Fig. 14:

• Remove from the walls surfaces any plaster and all the inconsistent parts in order to obtain a compact and mechanically resistant support that does not lead to the detachment of the subsequent applications;
• On this substrate's dry surface, previously saturated with water (condition s.s.a) to avoid the absorption of the mortar by capillarity of the bricks, must by applied a first uniform layer of plaster;
• The bidirectional dense textured net, in our case made of basaltic fibers, has to be applied widely on the whole walls surfaces. It must be incorporated partially in the fresh mortar, foreseeing an overlap length (Ls) of the net bands for about 20 cm in order to guarantee the mechanical continuity of the intervention;



a) Cross-section: in green are characterized the F.R.C.M. applied at the mezzanine floor; on the left side there is a technical detail of the reinforced plaster with fiber transversal connector



(b) Sequence of the different work phases in elevation; L_s is the length of overlap for the net bands

Fig. 14 Improvement intervention with F.R.C.M. and its application procedure

Table 7 Values of the improved mechanical parameters obtained from the average ones according to the requirements of Table C8.5.II

Wall type	f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
MBI	480	11,4	2250	750	18

Table 8 Outcomes of the pushover analysis for the model at LC2 with improvement interventions

Dir.*	Seismic Load distribution	Ecc. [cm]	α_{SLC}	α_{SLV}	α_{SLD}	α_{SLO}
+X	Static Forces	234.6	1.034	0.929	1.491	1.692
-Y	Static Forces	296.5	0.713	0.641	1.559	1.350

* Out of a total of 24 different types of pushover analysis conducted for each model, here are reported only the heaviest ones for each seismic direction

- Execution in compact areas of the masonry of drills for the realization of transversal connections, n° 4 - 5 every m^2. Those are realized by the insertion of a transversal bar and by the coupling to the net of fiber connectors terminal tufts. This enhancement provides artificial masonry through stones without increasing the final thickness of the intervention even in correspondence of the connector itself;
- A final protective and architectural layer plaster must be applied moreover to incorporate the support and to close any gaps. The total thickness of this strengthening is about 2 cm.

6.3 Non-linear static analysis in LC2 with strengthening in F.R.C.M.

In order to evaluate the incidence of those next-gen composite materials'application regarding the prevention of the dangerous in plane wall's collapse a supplementary global analysis has been conducted (Meireles *et al*2014). Always starting from the same model and settings, it was deemed appropriate to model numerically the aforesaid structural-strengthening within the possibility, offered by the Italian Building Code (NTC 2018) of improving the mechanical characteristics of the walls involved by structural consolidation interventions using appropriate meliorative coefficients. This choice allowed to match the F.R.C.M. with the large item of the reinforced plaster and to take into account the presence of the transversal connectors and the improving of the joints at the corners between orthogonal masonry panels it was decided to remove the stiffness reduction previously introduced because of the cracks; in Table 7 are proposed the improved mechanical parameters which were later divided by FC equal to 1.2 in order to obtain design values. This choice, moreover, reflects the variation of the drift associated to the wall which, as result of this type of intervention, would significantly increase its ductility in the plane.

As previously mentioned, this improved material has interested only the internal wall constituting the mezzanine floor but the results have been evaluated still in terms of global seismic response of the whole structure, Tab. 8. Most of the remaining analyses show values of $\alpha_{SLX} \geq 1$ showing, overall, a slight improvement to the–structural response especially in the X direction of the earthquake.

7. Conclusions

Nowadays, thanks to the new technological tools, the seismic assessment and improvement methods must look at the structural problems from a wider perspective, recognizing the quality of the architectural object in question and acting accordingly. In the present contribution we dealt with the seismic global evaluation of the heritage building designed by the architect Luigi Vanvitelli, located in Perugia, and headquarters of the homonymous university. In this framework the FME behaviour was intended as a research tool useful for understanding the structural behaviour and seismic vulnerability of this historical and iconic building. Three types of analysis were carried out, the preliminary one with low level of knowledge (LC1) needed to identify the weakest elements of the structure and to direct the implementation of the subsequent surveys and experimental tests. Characterized, later, the knowledge of the building with a higher level (LC2) it was possible to evaluate the actual safety of the masonry building recognizing the good functioning of the existing structures and attesting Palazzo Murena in a state that deserves anyway to be improved. Finally, the outcomes of this assessment have allowed to envisage future strengthening interventions aimed at the areas of proven structural weakness. The comparison was carried out in terms α_{SLX}; in the transition from LC1 to LC2 a percentage variation, regarding the average α_{SLV}, of 15% was recorded and from current status to the improvement intervention with FRCM an increase always in the average α_{SLV} was observed of about 20%; and it is worth noting that this last outcome was achieved without adding substantial weights to the structure. These results are promising but, as suggested by the last decree and even highlighted in this work, it will be necessary to further study the risks of collapse related to the activation of local mechanisms especially outside the wall's plane.

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