



Seismic Assessment of the Palace of Priors in Perugia

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Seismic assessment of the Palace of Priors in Perugia

Giulio Castori¹, Romina Sisti², Antonio Borri³, Marco Corradi⁴ and Alessandro De Maria⁵

Abstract. The seismic assessment of historical structures necessitates considering conservation and safety objectives as well as the possible presence of cultural heritage assets. To this end, this paper emphasises the results of a seismic evaluation procedure carried out by the authors on an illustrative case of study, the Palace of Priors in Perugia, that in addition to being one of the most important local governor buildings built, during the High Middle Ages, by Italian communes of Central Italy, it is characterized by the presence of a lot of artistic assets as well. Within this context, strong emphasis was placed on the seismic risk assessment of the structure carried out with reference to the Italian guidelines for heritage protection and conservation. More specifically, the paper investigates and critically discusses the seismic response of the building by using 3 different types of evaluation: territorial level analysis (LV1), local level analysis (LV2) and global level analysis (LV3).

Keywords: Numerical modelling; Historical masonry buildings; Nonlinear static analysis; Kinematic analysis.

1 Introduction

Due to the growing attention about the conservation of monumental masonry constructions, there is a compelling need for new strategies for the classification and analysis of ancient masonry structures and single structural elements, such wall panels, vaults, columns and buttresses and non-structural assets, such frescos, decorations, statues, etc. Because historic constructions may give a significant contribution to knowledge and understanding of the past, substantial harm to, or loss of, the significance of a listed structure should be always avoided when works are proposed for development-, conservation- or presentation-related purposes. Seismic strengthening interventions works should be characterized by a negligible impact on the historical significance of the building, meeting the requirements of the ‘minimum intervention’. Minimum intervention may be considered as a philosophy of designing restoration and reinforcement works with the characteristics to be: reversible, minimally invasive and historical significance-friendly [1].

Conservation bodies don’t easily allow both partially destructive and destructive testing campaigns [2][3] and structural designers can only rely on nondestructive testing to study the behavior of listed constructions (ground penetrating radar, videoendoscopic survey, infrared thermography, tomographic imaging, etc.).

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However, it is recognized that little increase in knowledge and understanding can be achieved using non-destructive techniques without conducting a numerical simulation. In this context, it is worth noting how the effectiveness of any numerical approach strictly depends by a comprehensive validation and calibration. Starting from the 90s, a great deal of research was thus conducted to evaluate the reliability of non-linear numerical investigations of masonry structures [4][5][6][7]. Validation is particularly important for historic masonry constructions, especially in earthquake prone areas since these numerical analyses are often used in many restoration and reinforcement projects, which have life-safety considerations.

The research presented in this paper is part of a national project (ARCUS), supported by the Italian Ministry of Cultural Heritage, Activities and Tourism (MiBACT), and it is aimed at investigating retrofitting interventions for Italy's earthquake risk museums, evidencing how an accurate assessment of a masonry monument or an historic building is a high priority. More specifically, the paper analyzes the results of a structural investigations performed on a medieval monumental building, the Palace of Priors in Perugia (Italy), that currently houses an important museum. Within this framework, the seismic risk of the structure was investigated using 3 different types of seismic analysis (LV1, LV2 and LV3) [8][9][10]. The first type of analysis (LV1) was conducted using a territorial scale strategy and it consisted in a simplified assessment of the collapse acceleration of the building. The second type of analysis (LV2) was based on the kinematics theorems of the limit analysis (macro-element approach) performed to analyze the structural safety of single structural elements. The last type of analysis (LV3) was performed using a nonlinear static analysis (global analysis) of the entire structure under seismic loading.

2 Assessment method

As above mentioned, the main aim of the present paper was to investigate and critically discuss the seismic response of a medieval monumental building (the Palace of Priors in Perugia), by using a performance-based multi-scale strategy. To this end, 3 different types of seismic analysis have been considered: territorial level analysis (LV1), local level analysis (LV2) and global level analysis (LV3).

2.1 LV1 (territorial level analysis)

The seismic vulnerability assessment of the building at a territorial level (LV1) was made using a simplified model, outlined by the Italian Guidelines [9] to investigate the seismic vulnerability of palace and villas. The main assumption is that the attainment of life safety limit state (SLV) is caused by the in-plane failure of the masonry walls [11].

Taking into account the construction phases, the structure was divided into four structural units (Figure 1), each of which was analyzed independently of each other.

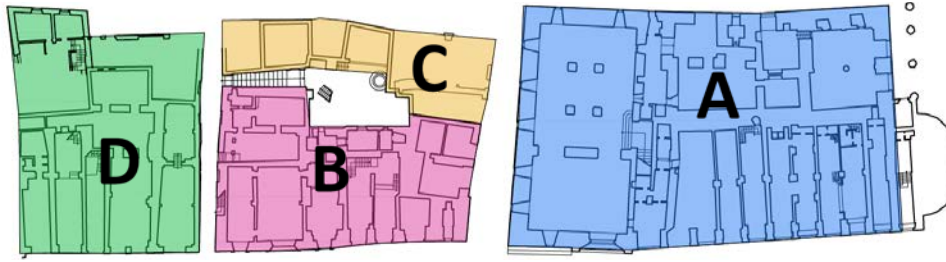


Figure 1. Structural units split up from the building.

The elastic response spectrum was evaluated according to the following formulation:

$$S_{e,SLV}(T_1) = \frac{qF_{SLV}}{e^*M} \quad (1)$$

where $q = 3.0$ denotes the behavior factor, T_1 represents the fundamental period of the building, M and e^* are the total seismic mass and the ratio of the participating mass, respectively. The evaluation of $S_{e,SLV}(T_1)$, as seen from (1), requires then to estimate the building shear capacity (F_{SLV}) as the lowest value among the shear strengths of masonry piers along the two main directions:

$$F_{SLV} = \min\{F_{SLV,xi}; F_{SLV,yi}\} = \min\left\{\frac{\mu_{xi}\xi_{xi}\zeta_x A_{xi}\tau_{di}}{\beta_{xi}\kappa_i}, \frac{\mu_{yi}\xi_{yi}\zeta_y A_{yi}\tau_{di}}{\beta_{yi}\kappa_i}\right\} \quad (2)$$

where μ_{xi} and μ_{yi} are coefficients that consider how the strength and the stiffness of masonry load-bearing walls are homogeneous along the two main directions, ξ_{xi} and ξ_{yi} are coefficients associated to the failure mode of masonry walls (equal to 1.0 for shear collapse mechanisms or 0.8 for compression-bending collapse mechanisms), ζ_x and ζ_y are assumed equal to 0.8 or 1.0 depending on the spandrel walls strength (weak or strong spandrel), A_{xi} and A_{yi} represent the shear resistant areas of the masonry panels, τ_{di} denotes the shear strength (design value) of each masonry panels, β_{xi} and β_{yi} are plan irregularity factors and κ_i is the ratio between the seismic loads at the i^{th} floor and total seismic load.

Table 1 shows the results of the analysis: once the elastic response spectrum has been evaluated ($S_{e,SLV}$), the corresponding return time (T_R) is calculated and the acceleration corresponding to the achievement of the SLV state (a_{SLV}) is finally evaluated. The acceleration factor ($f_{a,SLV}$), defined as a ratio between a_{SLV} and the reference acceleration for the SLV state ($a_{g,SLV}$), is a representative parameter of the structure's behavior.

Table 1. LV1 analysis results.

Structural unit	Storey	$F_{SLV,x}$ [KN]	$F_{SLV,y}$ [KN]	F_{SLV} [KN]	$S_{e,SLV}$ [m/s ²]	a_{SLV} [g]	$f_{a,SLV}$
A	1	19625	24261	14530	2.098	0.094	0.439
	2	18128	14530				
	3	177703	15264				
	4	14609	20315				
	5	28762	31365				
B	1	8213	10660	7732	2.615	0.104	0.486
	2	7732	8753				
	3	10013	8309				
	4	13257	8842				
C	1	3765	2131	2131	2.165	0.069	0.322

	2	3380	3144				
	3	5485	4403				
D	1	4881	14980	4881	1.638	0.067	0.313
	2	6377	12489				
	3	7391	9720				
	4	10445	11356				

2.2 LV2 (local level analysis)

The assessment of the seismic vulnerability at a local level (LV2) required a preliminary analysis of the local failure mechanisms, which may occur in the building. At this evaluation level, it was necessary to identify a wide set of potential collapse scenarios, which, involving only single structural elements (incapable to transfer the seismic loads to the rest of the structure [12]), wouldn't be considered using a global analysis (Equivalent Frame method, EF). To this end, it was decided to use a macro-element approach based on the theorems of limit analysis (Rigid Macro-Block method, RMB). The construction was thus idealised as a system of elementary substructures, identified either by analyzing their structural characteristics (such as effectiveness of the existing wall-to-wall or wall-to-floor connections, constructive phases, etc.) or by considering the damaging effects of ground shaking observed, in the past, in similar structures (the damage pattern consequent to past earthquakes can facilitate the prediction of possible failure mechanisms, [13][14]).

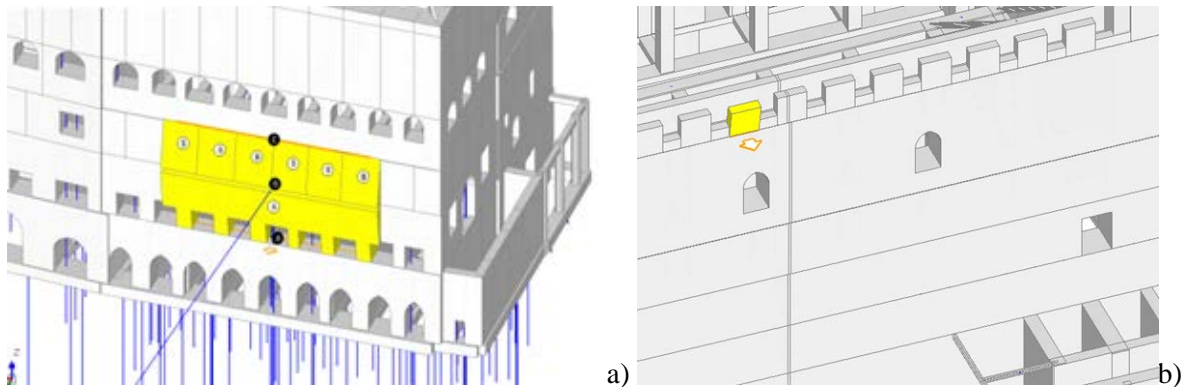


Figure 2. a) Mechanism of vertical bending of the lateral wall (Mec-01); b) Mechanism of overturning of the unlinked masonry portions (merlons, Mec-02).

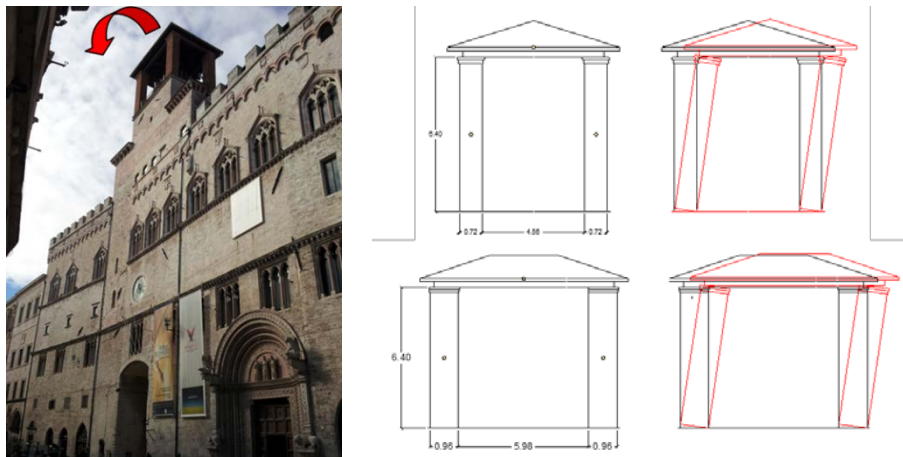


Figure 3. Mechanisms of overturning of the bell tower (Mec-03).

In such a context, a number of potential local mechanisms (mostly out-of-plane ones, as shown in Figure 2 and Figure 3) were selected and the corresponding values of the seismic activation multiplier (λ) were evaluated according to the methodology (Theorem of Virtual Works) proposed by the Italian code [15]. These values were then used to assess the seismic acceleration (spectral acceleration, a_0^*), responsible for the onset of the selected mechanisms, through the following expression:

$$a_0^* = \frac{\lambda g}{CF \cdot e^*} \quad (3)$$

where g represents the acceleration of gravity, CF denotes the confidence factor whereas e^* the ratio of participating mass.

Finally, the seismic analysis was performed through the use of the I_{ss} index (Seismic Safety Index), evaluated as the ratio between the maximum value of the acceleration (seismic capacity, $IM_{cap} = a_0^* q$, where a_0^* is given by (3) and $q = 2.0$ is the behavior factor according to [16]) compatible with the fulfillment of the SLV state and the reference target value of the seismic demand (IM_{dem}) given by:

$$IM_{dem} = \begin{cases} a_g S & \text{for mechanisms at ground level} \\ \max(a_g S; S_e(T_1) \psi(Z) \gamma) & \text{for mechanisms at a certain heights} \end{cases} \quad (4)$$

being: a_g (0.211g) the seismic spectral acceleration on stiff soil, S (1.433) a factor depending on the ground type (B type), S_e the spectral acceleration for the elastic design (at $T = T_1$), $\psi(Z)$ a normalized function that describes the amplitude of the I° natural mode of the building; γ a modal participation factor.

Table 2. LV2 analysis results.

	Seismic capacity [IM_{cap}]	Seismic Safety Index [I_{ss}]
Mec-01	0.213g	0.704
Mec-02	0.159g	0.528
Mec-03	0.127g	0.420

Results from the LV2 analysis are summarized in Table 2. As suggested by [9], it can be noted how, even if I_{ss} values less than 1.0 should indicate the mechanism is not verified, lower values of the minimum level of safety (up to 0.780) are acceptable if it is proved that the required strengthening works (necessary to increase the value of Seismic Safety Index) are inconsistent with the conservation and preservation requirements. Within this approach, by analyzing the I_{ss} values it is worth pointing out how the overturning mechanisms (Mec-02 and Mec-03) show a critical scenario, with values of the I_{ss} index considerably lower than the minimum level of safety outlined by the code. Even the mechanism Mec-01 (mechanism of vertical bending) showed a critical situation, but a value of the safety index significantly higher (0.704).

2.3 LV3 (global level analysis)

The seismic vulnerability assessment of the building at a global level was made through the use of a FE model, able to evaluate the values of seismic acceleration that leads the entire structure to a given performance level.

A nonlinear static analysis (pushover analysis) of the entire building, subjected to a monotonically increasing pattern of inertial forces (representing the in-plane forces that would be experienced by the structure under seismic loading), was used to investigate its seismic response [17]. More specifically, the structure was modelled through the use of an equivalent framed system composed of beam elements, able to simulate the behavior of both the spandrel walls and the wall piers (Figure 4a). As suggested by the Italian Code [16], two sets of horizontal forces, both depending on the mass distribution, were then applied (not simultaneously) along the two main directions (x and y) of the building: the first load distribution (modal distribution) was directly proportional to the corresponding displacements of the fundamental period of the structure, while for the second (uniform distribution) it was assumed a distribution proportional to inertial masses.

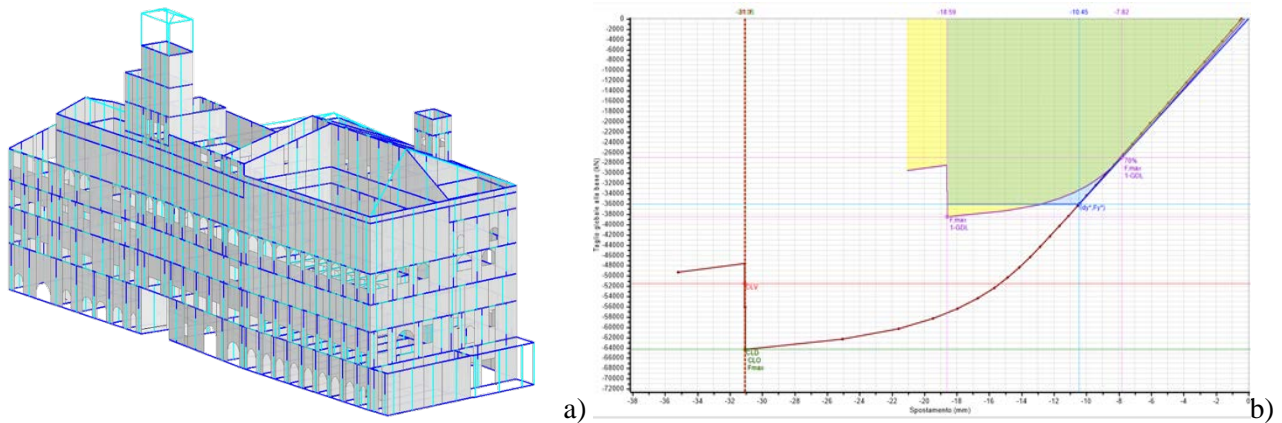


Figure 4. LV3 analysis: a) Equivalent Frame Model; b) Capacity curve in the x-direction.

At this evaluation level, the structure was modelled assuming the masonry material to be isotropic and making use of the mechanical parameters (which represent conservative estimations of the average values of the most common masonry typologies) provided by the Code [15]. Accordingly, the shear (G) and Young (E) moduli were assumed equal to 860 (410) MPa and 1080 (1230) MPa, for solid brick (stone) masonry respectively, while the shear and compressive strength were assumed equal to 0.060 (0.035) MPa and 2.4 (2.0) MPa, for solid brick (stone) masonry respectively.

Finally, as suggested by the Code [15] three different failure modes were considered for both the spandrels and wall piers: “rocking” failure, diagonal shear cracking and shear sliding failure. The total shear at the base of the structure vs the displacement of a selected control point (placed on the center of mass at the roof level) is shown Figure 4b. Such a diagram represents the capacity curve of the construction as it gives the maximum value of the shear force that is bearable by the structure, i.e. the value of the Peak Ground Acceleration (PGA) responsible for the onset of the I° collapse mechanism.

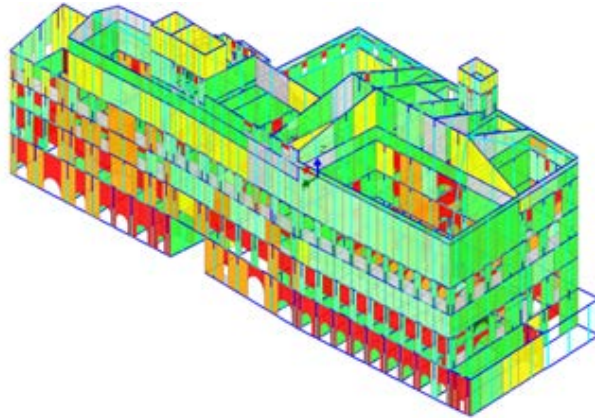


Figure 5. LV3 analysis results: cracking pattern.

Varying the loading conditions and performing an analysis for each load distribution (modal and uniform distribution), it was possible to observe how the most severe loading condition was in the transversal direction (x-direction) for the uniform distribution. Referring to the results of such an analysis (obtained through the use of the Aedes Pcm software [18]), in this direction the structure exhibited a poor performance (the analysis stopped at a value of the total shear equal to approximately the 50% of the building's overall weight), as a result of the progression of the out-of-plane mechanisms, due to the bending moment (Figure 5), of masonry piers in longitudinal direction (y-direction). As in the case of LV2 analyses, the seismic risk of the structure was synthetically investigated through the use of the I_{ss} index (Seismic Safety Index) by comparing the maximum value of the Peak Ground Acceleration (seismic capacity, $IM_{cap} = 0.154g$) responsible of the achievement of the assumed performance level (SLV state) to the reference target value ($IM_{dem} = 0.302g$) of the seismic demand (always in terms of PGA) provided by the code. The corresponding value of the seismic safety index ($I_{ss} = 0.509$), being significantly lower than the minimum level of safety (0.780) required by the Italian Guidelines [9], clearly indicate how the structure is not verified.

3 Conclusions

The research presented in this paper is aimed at discussing and validating a seismic evaluation procedure carried out by the authors on an illustrative case of study, the Palace of Priors, a medieval monumental building located in Perugia (Italy). To this end, the seismic risk of the masonry structure was investigated using a performance-based multi-scale strategy, based on 3 different types of seismic analysis: territorial level analysis (LV1), local level analysis (LV2) and global level analysis (LV3).

The first type of analysis (LV1), conducted using a territorial scale strategy, has evidenced a critical situation with values of the acceleration factor ($f_{a,SLV}$) ranging from 0.313 to 0.486. The second type of analysis (LV2), based on the kinematics theorems of the limit analysis and performed on a wide range of potential collapse scenarios, has highlighted several structural deficiencies in the out-of-plane response of the structure, mainly related to the identification of overturning mechanisms of single structural elements. Lastly, the third type of analysis (LV3), conducted using an equivalent frame approach, has allowed to highlight how, assuming a box-

like behavior, the damage was mainly associated with the progression of the out-of-plane mechanisms (due to the bending moment) of masonry piers in longitudinal direction. In spite of an intrinsic coherence of the 3 different types of seismic analysis, a clear trend line from the attained results has not been derived. Such a conclusion should highlight the importance of a critical approach to the problem, encouraging the adoption of different types of analysis to reduce the negative effects of unavoidable unknowns that affect the seismic response of a monumental building.

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