

Seismic safety assessment of a masonry building according to Italian Guidelines on Cultural Heritage: simplified mechanical-based approach and pushover analysis

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Abstract The seismic safety assessment of a case study of a masonry building located in Naples (Italy) was performed together with a critical appraisal of the methods used. Being built before the introduction of proper seismic code provisions, this unreinforced masonry building could be representative of many other vulnerable historic buildings in earthquake-prone urban areas. First, a simplified model of the global seismic response was analyzed according to the LV1 assessment level provided by the Italian Guidelines on Cultural Heritage. The results obtained using old and updated versions of these guidelines were compared. A good agreement was revealed with reference to the detection of the weaker direction and the prevailing failure mechanism, but some differences were found about the calculation of the base shear capacity and the corresponding ground acceleration. Then, the achieved results were compared with those obtained using a more refined approach of nonlinear static analysis according to the LV3 assessment level. The results were reported in terms of damage and collapse mechanisms of masonry walls, pushover curves and seismic safety indexes. A further comparison was carried out between the model with flexible horizontal structures and that with the assumption of all the floor diaphragms as completely rigid. Although the two assessment methods LV1 and LV3 are not alternative to each other, since belonging to two different levels of evaluation, some critical issues were addressed in order to derive useful information on the reliability and the limits of validity of the simplified mechanical model, characterized by a force-based approach.

Keywords Masonry structures · Simplified mechanical-based approach · Safety levels · Global seismic response · Pushover analysis · Rigid and flexible floors

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1 Introduction

The necessity of technical documents for the seismic risk assessment and seismic retrofitting of cultural heritage is generally stressed by the knowledge that earthquake represents one of the main cause of damage to constructions and many interventions carried out in the past have been resulted as ineffective or dangerous, being often executed without having the necessary knowledge of the real structural behaviour.

In recent years, several researchers have concentrated their interest on cultural heritage constructions spread across European countries, also stressing the advancement of technical standards (Giuffrè and Carocci 1996; Salonikos et al. 2003; D’Ayala and Speranza 2003; Lagomarsino et al. 2004; Roca et al. 2005; Binda et al. 2006; Lourenço et al. 2007; Borri and De Maria 2009; Andreini et al. 2013; Giresini 2015; Sassu 2006), while the force-based methods of analysis have been progressively abandoned in favour of performance-based approaches (Priestley 2000; D’Ayala and Ansal 2012; Lagomarsino and Cattari 2015).

The theme of the evaluation and the reduction of the seismic risk of historical constructions is becoming more and more relevant in any earthquake-prone area and especially in Italy, due to the huge number of potentially vulnerable heritage structures. In this country a number of Codes and Guidelines have recently been produced, some of which are worthy of consideration:

- OPCM n. 3274 (2003), updated with OPCM. n. 3431 (OPCM 3431 2005);
- Technical Rules for Constructions, issued in 2008 [shortly named NTC08 (NTC 2008)];
- Instructions for the application of the New Technical Rules for Constructions (shortly named Circular n. 617/2009 (MCIT 617 2009));
- Guidelines about the preservation of historical and architectural heritage: “Seismic risk evaluation and reduction of the cultural heritage” [herein named “old Guidelines” or Pre-NTC08 (DCCM 2007)] and “Seismic risk evaluation and reduction of the cultural heritage, with reference to the Technical Rules for Constructions” [herein named “new Guidelines” or Post-NTC08 (DCCM 2011)]. The difference in the two versions also reflects the updating of the hazard maps provided in the Italian Structural Code NTC08 (NTC 2008).

The whole of these standards and rules, more advanced than the Eurocodes dealing with these arguments (Eurocode 8 2005; Borri and De Maria 2009; Magenes and Penna 2009), is a legacy of knowledge deriving from the Italian seismic recent experiences (just like the Umbria earthquake of 1997, the Molise earthquake of 2002 and the Abruzzo earthquake of 2009).

In particular, the Italian Guidelines on Cultural Heritage introduce a new approach for the evaluation of the seismic safety through a multilevel path which tries to coordinate the phases of investigation and assessment with different grades of resolution depending on different possible objectives. In particular, three levels of investigation and assessment are defined:

LV1: territorial-scale seismic evaluation through a simplified mechanical-based approach;

LV2: seismic evaluation to be used in case of local interventions on a building;

LV3: accurate evaluation of the seismic safety of a building.

The first level assessment, LV1, is oriented to highlight, on a regional scale, critical situations in terms of seismic vulnerability and to provide a classification of risk and a

priority list for further investigation aimed at the conservation of the architectural heritage. Adopting a force-based approach, this level relies on a simplified structural model that requires integrating a limited number of geometrical and mechanical parameters with qualitative data derived from visual tests, construction features and stratigraphic survey. The second level assessment, LV2 is aimed at evaluating the seismic safety when local interventions on single parts of a building are carried out. It is important to underline that the LV2 can be used only when local interventions do not modify the structural behaviour of the building. Lastly, the LV3 is based on the global structural response of the building in order to define the values of acceleration leading the structure to each limit-state. In this case, the displacement-based approach is adopted, for which the global behaviour is governed by the in-plane capacity of the walls discretized in panels where the nonlinear response is concentrated.

The seismic safety is evaluated for each level by an index summarizing the comparison between the expected seismic demand and the seismic capacity.

It is worth highlighting that the LV1 and LV3 are based on simplified and accurate global models, respectively, both provided by the combined effect of floor diaphragms and in-plane response of structural walls. This means that they are directly comparable. Depending on the specific features of the building, the LV3 could also be performed by extending the modelling approach proposed for LV2 in a systematic way to all the macro-elements that compose the building. On the other hand, the LV2 allows the seismic assessment of the local failure modes, mainly due to the out-of-plane responses of walls. Local mechanisms may be modelled as kinematic chains of masonry portions, regarded as rigid macro-blocks or micro-block assemblages, interacting through interface elements. Specific literature is devoted to this issue, including some recent developments based on limit-state analysis and rocking rigid block dynamics (Casapulla et al. 2010, 2014, 2017; Casapulla 2001, 2015; Casapulla and Maione 2016a, b, 2017a, b; Casapulla and Argiento 2016, 2018; Lagomarsino 2015; Giresini et al. 2016; Giresini and Sassu 2017).

In this paper, the seismic assessment of a 19th century masonry building located in Naples (Italy) is carried out by exploring the two levels of evaluation LV1 and LV3 provided by the Italian Guidelines on Cultural Heritage. Being built before the introduction of proper seismic code provisions, this unreinforced masonry building could be representative of many other vulnerable buildings in historic centres, despite the enormous diversity of structural forms and materials from country to country. It is well known, in fact, that the structural assessment of such buildings is a crucial issue in areas with a moderate to high seismic hazard like many earthquake-prone areas in Europe and other countries.

The main objective of this work is to verify the reliability and the limits of validity of some aspects of a simplified mechanical model, characterized by a force-based approach, through its comparison with a more sophisticated model, which takes into account the nonlinear behaviour and the ductility reserve of the masonry structures in the post-elastic range. The paper is particularly focused upon testing the performance of the synthetic indexes of vulnerability proposed by the Guidelines and their capability to provide information in agreement with the structural behaviour of the buildings. It is worthy to clarify that the objective is not to assess the validity of the LV1 method as a whole, but with reference to a case study to test the capability of the assumed simplified mechanical model to predict the main issues characterizing the seismic behaviour of the building. The results obtained in terms of structural response do not have a statistical meaning because they are referred to a single building. Moreover, these results could not depend on the

number of the analyzed buildings, as they would be strongly influenced by the compliance of the case study with the requirements of regularity implied by the theoretical model.

The procedure (Pre-NTC08) implemented on line by MiBACT (Italian Ministry of Cultural Heritage and Activities and Tourism) and named SIVARS system (www.benitutelati.it) is used for the LV1 and the results are compared with those calculated with the improved procedure referred to the current Italian Code (Post-NTC08). The approach of nonlinear static analysis is instead adopted for the LV3 level assessment by using TREMURI computer program (Lagomarsino et al. 2013), based on the analysis of an equivalent three-dimensional frame with nonlinear behaviour.

It is worth highlighting that, even if the information and discussions in this paper are referred to a specific case study of Italian masonry building by using Italian codes, some aspects and results could be of general interest for the international scientific community and practitioners. The presented analysis could potentially be extended/applied to other unreinforced masonry buildings in different countries by using different codes. In fact, the findings of this study give a useful insight in the main differences between force-based and displacement-based approaches which are still being used in seismic prone areas to assess the building safety.

2 The case study: “Pelella Palace”

“Pelella Palace” is placed at the corner of an urban block located in Afragola (Naples, Italy) (Figs. 1, 2, 3) and is characterized by a “C” plan (35.87 m × 11.15 m) with three storeys above ground level and one attic, for a total height of 16.2 m. The main entrance,

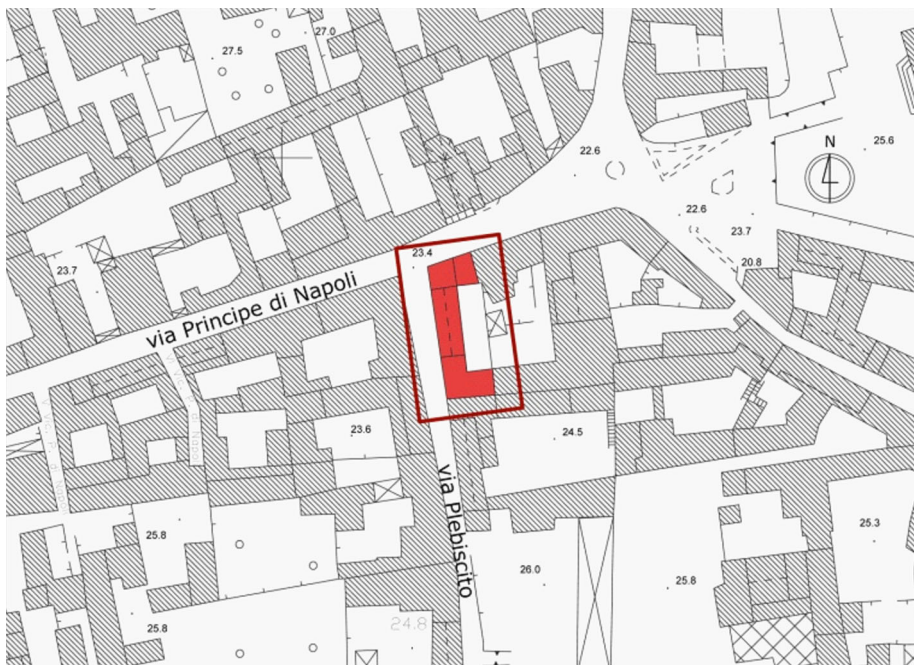


Fig. 1 Location of the “Pelella Palace” in the urban context

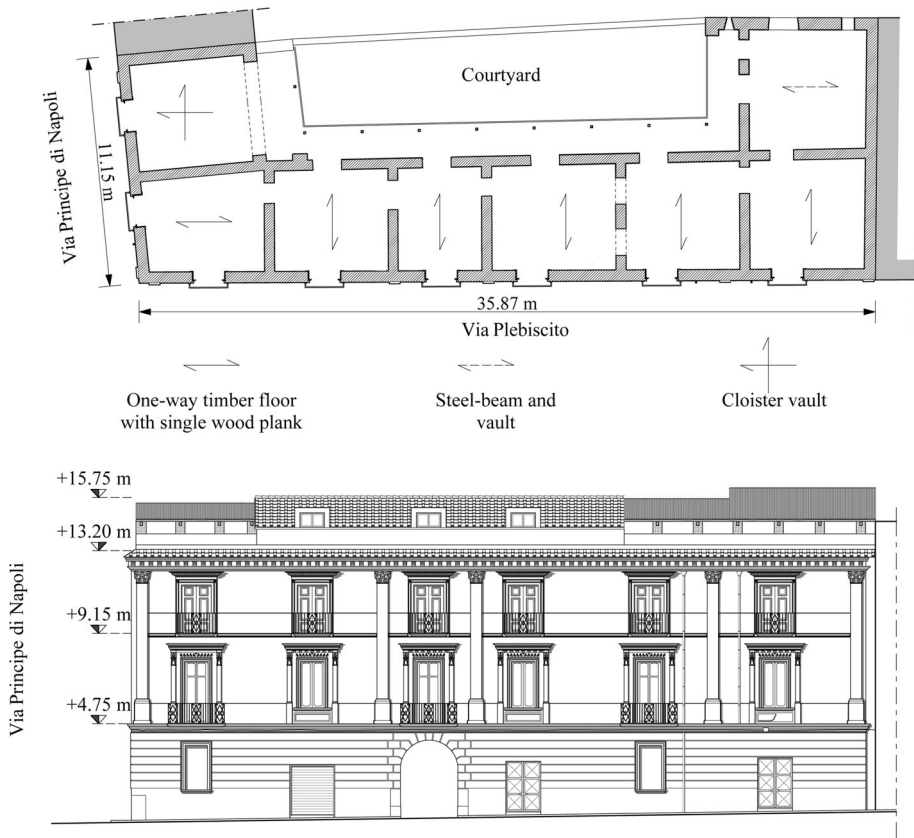


Fig. 2 Plan and elevation layouts of the “Pelella Palace”



Fig. 3 External views of the “Pelella Palace”

located on the long façade, leads to a rectangular courtyard in the rear of the building; a typical Neapolitan “open staircase”, with sloping vaulted ceilings and dome cover, serves the first and second floors. It was built in the end of 19th century with traditional construction techniques of the area, i.e. yellow tuff stones and traditional mortar for the

vertical structures. The tuff wall typology is a three-leaf wall, with two outer shells and a thick inner core of rubble material.

The horizontal structures are made out of timber floors, with the exception of two barrel vaults on the ground floor (one with lunettes), one cloister vault covering the staircase and two diaphragms in the south-east area composed by steel beams and short-span brick vaults. A representative plan of the third floor with the orientation and type of the floor diaphragms is reported in Fig. 2. The original roof was made of timber beams covered by a layer of tiles. A second type of coverage, added to the original at a later time, is made out of timber beams and asbestos.

Even if the case study building is included in an aggregate block, it is herein analysed as an isolated structure in order to allow the comparison among the different approaches, taking into account that the LV1 method still does not refer to aggregate buildings. This assumption is also supported by the urban position and the morphological configuration of the building which can actually be regarded as a well defined structural unit with substantially autonomous behaviour; in fact, due its corner position, only a weak confinement action is exerted on it by the adjacent buildings. One of these is a RC frame building with infill walls and some lateral space from the building under study; the masonry building on the other side, instead, has approximately the same height of the “Pelella Palace” and the orientation of its diaphragms is assumed to avoid interaction with the adjoining walls. However, although beyond the scope of this paper, a rigorous approach would require a detailed analysis of the entire aggregate, also taking into account the potential interactions due to the structural contiguity within the aggregate.

Despite some phenomena of damage and decay, the building appears not to be affected by local out-of-plane failure modes and this could allow assuming a box-like behaviour. Actually, although the analysis of local failures should be performed in the context of an exhaustive seismic verification, it is disregarded in this paper because the attention is only focused on the global seismic response. This is evaluated by means of two models with different levels of investigation and assessment according to the Italian Guidelines for cultural heritage. Both models are based on the combined effect of floor diaphragms and in-plane response of structural walls (box-like behaviour).

On the other hand, the quantification of the mechanical parameters of the structural elements depends on the knowledge of the construction. In fact, based on three levels of knowledge (LC1, LC2 and LC3), implying geometrical mapping, experimental investigation and historical research, the confidence factor F_C represents the material safety factor to be used for the seismic evaluation. An interesting discussion of the issues pertaining to the knowledge of the building can be found in the paper by Binda and Saisi (2009).

In the absence of specific experimental data for the building under study, the values of the mechanical parameters of tuff stone masonry suggested by the Circular n. 617/2009 (MCIT 617 2009) for the minimum knowledge level LC1 are assumed as a reference. However, some improving features of the masonry are taken into account, such as the presence of mortar of good quality and effective transverse connections between the outer leaves (headers). These qualitative features allow amplifying the reference values with the corrective coefficients reported in Table 1. Hence, the design values of the strength parameters are obtained introducing a confidence factor $F_C = 1.35$, while a further corrective coefficient equal to 0.5 is assumed for the elastic stiffness properties in order to take into account cracking phenomena, according to Lagomarsino et al. (2013). In Table 1 the design values of the elastic moduli, E and G , and the masonry compressive and shear strengths used in the analysis, respectively f_m and τ_{0d} , are reported together with their reference values and corrective coefficients.

Table 1 Mechanical properties of tuff masonry

| Mechanical parameters | Reference values for LC1 (N/mm ²) | Corrective coefficients (masonry qualitative features) | Design values (N/mm ²) |
|-----------------------|---|--|------------------------------------|
| E | 1080 | 1.5 | 810 ^a |
| G | 360 | 1.5 | 270 ^a |
| f_m | 1.40 | 2.25 | 2.33 ^b |
| τ_{0d} | 0.028 | 2.25 | 0.047 ^b |

^aValues accounting for the cracked condition

^bValues accounting for the confidence factor F_C

The necessity of introducing the corrective coefficients is an attempt to represent the enormous variability of structural forms and materials that can be found in historical constructions. The definition of qualitative criteria appears essential in cases where the in situ direct experimental measurement of material parameters is not viable or considered unreliable. However, the application of multiple/null corrective coefficients could also lead to overestimation/underestimation of such parameters which can relatively affect the overall capacity of the building. Therefore, a more accurate criterion would be required and the approach recently proposed by Borri et al. (2015) through the Masonry Quality Index method looks interesting. This method consists in evaluating the presence, the partial presence, or the absence of certain parameters that define the “rule of the art”, namely a set of construction devices that, if executed during the construction of a wall, provide a good behaviour and ensure its compactness and monolithicity.

2.1 Reference seismic actions

In order to assess the seismic demand, the Italian Code NTC08 (NTC 2008) requires to define the reference lifetime V_R of the building, depending on its use and nominal lifetime. The concept of nominal lifetime for cultural heritage can be interpreted as the period of validity of the safety assessment, beyond which it is necessary to conduct a new evaluation and/or upgrading. The assumption of a very long nominal lifetime, as in theory would be required for the cultural value of the architectural work, would result in the definition of severe reference seismic actions and invasive interventions to improve the seismic response. In fact, the return period of the seismic action depends on the reference lifetime of the construction. Taking this into account, for the building under study it is assumed the nominal lifetime $V_N = 50$ years and a use factor $C_u = 1$ corresponding to a frequently used building with normal crowding (Class II) according to the classification adopted by the Guidelines for the cultural heritage (DCCM 2011). Hence a reference lifetime $V_R = C_u \times V_N = 50$ years is defined. According to the Post-NTC08 Guidelines and with reference to the life-safety limit-state (SLV), the seismic action is evaluated assuming the excess probability $P_{V_R,SLV}$ of 10%, as for ordinary buildings. This parameter, together with V_R , leads to define the reference return period (Annex A of NTC08):

$$T_{R,SLV} = -\frac{V_R}{\ln(1 - P_{V_R,SLV})} = -\frac{50}{\ln(1 - 0.1)} = 475 \text{ years} \quad (1)$$

Moreover, a soil type B (deposits of very dense sand, gravel, or very stiff clay) is assumed according to surveys made in the area, to which corresponds the soil factor $S = 1.2$ accounting for topographic and stratigraphic conditions (Laudiero 2012). The fundamental period T_f of the building under study is evaluated through the simplified formula of NTC08 as a function of its total height H :

$$T_f = 0.05H^{3/4} = 0.4037 \text{ s} \quad (2)$$

The parameters of the seismic hazard related to the site of Afragola ($40^\circ55'37''20$, $14^\circ18'42''12$) in Naples, for a soil type B, are reported in Table 2. According to Annex B of NTC08 these parameters, corresponding to different return periods for SLV limit-state, are the following: $a_{g,SLV}$ is the peak ground acceleration (PGA) on type A site class expressed in g (gravitational acceleration); F_0 is an amplification factor; T_C is the period related to the upper limit of the constant spectral acceleration region; $S_{e,SLV}(T_f)$ is the acceleration ordinate of the elastic response spectrum relative to T_f . The latter is derived from the function of the spectral shape in the range of period of interest, i.e.:

$$T_B \leq T_f < T_C \quad S_{e,SLV}(T_f) = a_{g,SLV} \cdot S \cdot F_0 \quad (3)$$

where $T_B = T_C/3$ is the period related to lower limit of the constant acceleration region of the elastic response spectrum and the parameter S summarizes the stratigraphic and topographic features of the ground. This table is also useful for the calculation, by interpolation, of the seismic parameters corresponding to the capacity of the building, according to the procedure introduced by the Post-NTC08 Guidelines, as described in the following section.

3 A simplified mechanical-based approach for LV1 assessment level

The first level assessment (LV1) of the seismic safety of “Pelella Palace” is performed here through a simplified mechanical model taking into account the indications provided by the old and new versions of the mentioned Italian Guidelines (DCCM 2007, 2011) for “palaces, villas and other structures with bearing walls and intermediate floors”.

The model is based on a few geometric and mechanical parameters which are easy to acquire at a first stage of investigation and it does not allow taking into account the possible interactions with adjacent buildings.

Table 2 Seismic hazard parameters and spectral accelerations for SLV limit-state, relative to the site of Afragola (Naples) and soil type B corresponding to different return periods (NTC 2008)

| $T_{R,SLV}$ (years) | $a_{g,SLV}$ (g) | F_0 | T_C (s) | $S_{e,SLV}(T_f)$ (m/s^2) |
|---------------------|-----------------|-------|-----------|------------------------------|
| 30 | 0.046 | 2.339 | 0.404 | 1.297 |
| 50 | 0.060 | 2.35 | 0.435 | 1.693 |
| 72 | 0.072 | 2.345 | 0.449 | 2.039 |
| 101 | 0.085 | 2.354 | 0.456 | 2.410 |
| 140 | 0.100 | 2.352 | 0.462 | 2.809 |
| 201 | 0.117 | 2.346 | 0.467 | 3.308 |
| 475 | 0.166 | 2.389 | 0.475 | 4.715 |
| 975 | 0.209 | 2.460 | 0.479 | 6.133 |
| 2475 | 0.274 | 2.574 | 0.482 | 7.891 |

The assessment of the seismic capacity is based on the assumption that the structure exhibits a global behaviour with damage/collapse of the walls in their plan due to shear or bending. The procedure first involves the calculation of the shear strengths at each storey level, along two orthogonal x and y -directions (main axes of the building). For x -direction and floor i , for example, the following expression is assumed according to Post–NTC08 version:

$$T_{xi} = \frac{\mu_{xi} \cdot \xi_{xi} \cdot \zeta_{xi} \cdot A_{xi} \cdot \tau_{di}}{\beta_{xi}} \tag{4}$$

Without the coefficient ζ defined later in this section, the same expression is also valid for Pre–NTC08 version of the Guidelines. Hence, it is clear from Eq. (4) that the meaningful geometrical and mechanical parameters are the area A_{xi} of the resistant sections of masonry piers in the considered direction and the design value of the masonry shear strength τ_{di} . The latter parameter is a function of τ_{0d} (Table 1) and the average normal stress σ_{0i} acting at floor i , according to the formula (Pre and Post–NTC08):

$$\tau_{di} = \tau_{0d} \sqrt{1 + \frac{\sigma_{0i}}{1.5\tau_{0d}}} \tag{5}$$

The coefficients β_{xi} and μ_{xi} in Eq. (4) bring into account respectively the irregularities in plan and the uniformity of stiffness and strength of the piers and they can be derived by simplified formulas on the basis of geometric criteria. Some other aspects of the structural behaviour, instead, such as the prevailing failure mode for the masonry piers and the effectiveness of the spandrels, are evaluated on the basis of qualitative considerations and their influence on the global response is measured in a conventional way through the parameters ξ_i (for piers) and ζ_i (for spandrels); in fact, both these parameters assume a value of 0.8 in case of bending failure for piers associated to weak spandrels (cantilever behaviour), and a value of 1 in case of shear failure for piers and strong spandrels (shear-type behaviour). The parameter ζ , as anticipated, is introduced by the new Guidelines to improve the representation of the global structural behaviour, though still in a simplified manner.

According to these concepts and on the basis of the geometric surveys of the elevation layouts of the “Pelella Palace” in both directions, a greater tendency to shear-type failure can be predicted in x -direction (thick piers and spandrels) with the exception of the top floor with lightly overloaded piers and slender spandrels. It is worth highlighting that the “weak” or “strong” behaviour of the spandrels is here only related to their geometric shape and consistency with the two reference models of cantilever and shear-type behaviour, while they should actually be considered all “weak” due to the lack of tensile resistant elements coupled to them. The opposite condition can be predicted in y -direction, as reported in Table 3, where the values of all the other parameters required for the first level of assessment are reported for each storey level and each direction.

Hence, the base shear forces F_{SLV} corresponding to the activation of the shear resistances T are calculated for each direction and each floor. For x -direction and floor i , for example, it is:

$$F_{SLV,xi} = \frac{T_{xi}}{\kappa_{xi}} \tag{6}$$

where $\kappa_{xi} \leq 1$ is introduced by the new Guidelines as the ratio between the resultant of the shear forces in x or y direction related to the i -th floor and the corresponding base shear. Pre–NTC08 Guidelines, instead, assume $F_{SLV,xi} = T_{xi}$ and, hence, implicitly $\kappa_{xi} = 1$.

Table 3 Parameters required by the first level assessment (LV1)

| Floor | Parameters related to the floor | | Parameters related to two orthogonal directions | | x direction | y direction |
|------------------|------------------------------------|---------|---|---------|-------------|-------------|
| 1 | Storey height (m) | 4.80 | A_1 (m ²) | | 39.74 | 30.65 |
| | σ_{01} (kN/m ²) | 274.02 | μ_1 | | 0.82 | 0.80 |
| | τ_{d1} (kN/m ²) | 103.45 | ξ_1 | | 1 | 0.8 |
| | ϕ_1 | 0.32 | β_1 | | 1.03 | 1.25 |
| | | | ζ_1 | | 1 | 0.8 |
| | | | m_1 (kg) | | 356,393 | 357,170 |
| | | | κ_1 | | 1 | 1 |
| | Pre–NTC08 (SIVARS) | | $T_1 = F_{SLV1}$ (kN) | | 3272.64 | 1623.54 |
| | Post–NTC08 | | T_1 (kN) | | 3272.64 | 1298.83 |
| | | | F_{SLV1} (kN) | | 3272.64 | 1298.83 |
| 2 | Storey height (m) | 4.40 | A_2 (m ²) | | 38.34 | 25.25 |
| | σ_{02} (kN/m ²) | 190.88 | μ_2 | | 0.85 | 0.8 |
| | τ_{d2} (kN/m ²) | 90.09 | ξ_2 | | 1 | 0.8 |
| | ϕ_2 | 0.61 | β_2 | | 1.15 | 1.25 |
| | | | ζ_2 | | 1 | 0.8 |
| | | | m_2 (kg) | | 363,415 | 226,745 |
| | | | κ_2 | | 0.83 | 0.76 |
| | Pre–NTC08 (SIVARS) | | $T_2 = F_{SLV2}$ (kN) | | 2552.87 | 1164.69 |
| | Post–NTC08 | | T_2 (kN) | | 2552.87 | 931.75 |
| | | | F_{SLV2} (kN) | | 3075.90 | 1220.19 |
| 3 | Storey height (m) | 4.10 | A_3 (m ²) | | 36.21 | 25.09 |
| | σ_{03} (kN/m ²) | 101.54 | μ_3 | | 0.85 | 0.0 |
| | τ_{d3} (kN/m ²) | 73.05 | ξ_3 | | 1 | 0.8 |
| | ϕ_3 | 0.88 | β_3 | | 1.25 | 1.2 |
| | | | ζ_3 | | 1 | 0.8 |
| | | | m_3 (kg) | | 314,498 | 227,865 |
| | | | κ_3 | | 0.50 | 0.48 |
| | Pre–NTC08 (SIVARS) | | $T_3 = F_{SLV3}$ (kN) | | 1798.61 | 977.58 |
| | Post–NTC08 | | T_3 (kN) | | 1798.61 | 782.04 |
| | | | F_{SLV3} (kN) | | 3620.39 | 1645.71 |
| 4 | Storey height (m) | 1.6 | A_4 (m ²) | | 15.92 | 8.01 |
| | σ_{04} (kN/m ²) | 101.54 | μ_4 | | 0.8 | 0.91 |
| | τ_{d4} (kN/m ²) | 73.05 | ξ_4 | | 0.8 | 1 |
| | ϕ_4 | 0.88 | β_4 | | 1.07 | 1 |
| | | | ζ_4 | | 0.8 | 1 |
| | | | m_4 (kg) | | 52,429 | 26,379 |
| | | | κ_4 | | 0.08 | 0.06 |
| | Pre–NTC08 (SIVARS) | | $T_4 = F_{SLV4}$ (kN) | | 538.87 | 412.47 |
| | Post–NTC08 | | T_4 (kN) | | 431.23 | 412.47 |
| | | | F_{SLV4} (kN) | | 5456.44 | 7478.78 |
| Other parameters | | | | | | |
| M (kg) | T_f | e_1^* | e_2^* | e_3^* | q | $C(T_f)$ |
| 1923182.83 | 0.4037 | 0.25 | 0.84 | 0.86 | 3 | 2.5 |

The criterion to identify the global shear capacity of the building at the SLV limit-state is the same for the two versions of the Guidelines. It is in fact:

$$F_{SLV} = \min(F_{SLVxi}; F_{SLVyi}) \quad (7)$$

The assessment of the parameter κ_i requires assuming a mode shape characterizing the ultimate limit-state of the building. This represents a non trivial task when masonry buildings present irregular storey heights and distribution of masses, as it is for “Pelella Palace”. When the mode shape is not defined, the new Guidelines allow assuming a triangular shape for the horizontal displacements normalized to the unit value at the top of the building corresponding to a schematization of the first mode shape of a regular structure. However, whatever the mode shape, the more general expression of the parameter κ_i provided by the new Guidelines is:

$$\kappa_i = \frac{\sum_{j=i}^N m_j \phi_j}{\sum_{j=1}^N m_j \phi_j} \quad (8)$$

being m_j and ϕ_j , respectively, the mass along x or y direction and the horizontal displacement of the j floor. When a triangular mode shape is assumed and the floor masses m_j are constant together with the storey heights, the expression of κ_i only depends on the number N of floors, resulting:

$$\kappa_i = \frac{\sum_{j=i}^N j}{\sum_{j=1}^N j} \quad (9)$$

The informative system SIVARS implemented by MiBACT and accessible through the web by institutional authorization (www.benitutelati.it), is used to assess the shear strengths according to the old Guidelines, reported as “ F_{SLV} Pre–NTC08” in Table 3. The updated results including the missing parameters ζ and κ required by Eqs. (4) and (6) are, instead, reported as “ F_{SLV} Post–NTC08”. It is worth noting that, for each accounted direction and floor, the parameter κ reported in Table 3 is calculated according to Eq. (8), in consideration of the irregularities in the storey heights and masses distribution and of an assumed triangular mode shape. The actual first mode shape derived by modal analysis in Sect. 4 is very close to the triangular type.

From Table 3 it emerges that the two versions of the Guidelines are in good agreement on the identification of y -direction as the weaker direction and the evaluation of the floor–shear resistances T_i , that only differ for the parameter κ accounted only by Post-NTC08 version. In particular, the lowest floor–shear resistance is reported at fourth level and its value is $T_{y4} = 412.47$ kN for both versions of the Guidelines, being $\zeta_{y4} = 1$. This result is explained by the consistent reduction of the plan extension at the top level, which is, in fact, a rising-up rather than a full floor. On the other hand, sensible differences are reported in the evaluation of the global shear capacity F_{SLV} ; in fact, according to the criterion expressed by Eq. (7) the value provided by Post-NTC08 ($F_{SLV} = F_{SLVy2} = 1220.19$ kN) is almost three times the one provided by Pre-NTC08 ($F_{SLV} = F_{SLVy4} = 412.47$ kN). This discrepancy is basically due the parameter κ , introduced by Post-NTC08 version, as will be better discussed in Sect. 5.1.

From the shear capacity of the building, different values of ground acceleration and different safety indexes for the two versions of the Guidelines are also derived, as described in the following. For Pre–NTC08, the ground acceleration capacity with

reference to the SLV limit-state (a_{SLV}) is evaluated as the ratio between the shear capacity F_{SLV} (i.e. the base shear of the building) and the mass participating in the dynamic response:

$$a_{SLV} = \frac{q \cdot F_{SLV}(\text{Pre NTC08})}{e_1^* \cdot M \cdot C(T_f)} \quad (10)$$

where q is the behaviour factor, M and e_1^* are the seismic mass and the mass participation factor of the building, respectively, and $C(T_f)$ is a sort of amplification factor similar to F_0 . The choice of the behaviour factor q is a crucial aspect in linear modelling of masonry structures. Experimental tests performed by Tomažević et al. (2004) have shown that this parameter depends not only on the constructive system, but also on the mechanical properties and the regularity of the structural configuration. This finding is obviously relevant in particular for heritage buildings. Magenes (2006) has introduced an over-strength factor which takes into account the circumstance that the global failure is achieved for a value of the base shear higher than that corresponding to the failure of the first structural element; a similar approach is adopted by the Italian Code (NTC08) and provides values of the behaviour factor higher than those suggested by the Eurocode 8 (2005) for masonry buildings. For existing masonry buildings, in particular, the Post-NTC08 Guidelines suggest a range $2.8 < q < 3.6$, the higher value referred to buildings regular in elevation. For the same condition, the Circular n. 617/2009 (MCIT 617 2009) allows assuming a behaviour factor $q = 3$, without considering the regularity in plan. In light of these considerations, a behaviour factor $q = 3$ is assumed in the analysis.

As it comes to the mass participation factor, the old Guidelines propose two simplified formulas in function of the number of storey levels:

$$\begin{aligned} e_1^* &= \frac{N + 1 - k}{N} \\ e_2^* &= 0.75 + 0.25N^{-0.75} \end{aligned} \quad (11)$$

which are based on two recurrent failure modes of masonry buildings. The first formula is referred to a storey mechanism which may happen when the k -th floor is weaker than the others, so that the construction is unable to exploit the dissipative capability; the second formula takes into account a uniform collapse, which may happen with the failure of the horizontal spandrels and the bending failure of the piers at the base of the first storey level (this corresponds to the cantilever idealization). The first formula of Eq. (11) is used in this assessment, based on the choice of the storey mechanism which is associated by SIVARS to the top floor. The values of the parameters in Eqs. (10) and (11) assumed for this analysis are reported at the end of Table 3; they provide a ground acceleration capacity $a_{SLV} = 0.105$ g.

Therefore, according to Pre-NTC08 Guidelines, a sole safety index is identified as:

$$I_S = \frac{a_{SLV}}{\gamma_I \cdot S \cdot a_{g,SLV}} \quad (12)$$

where the denominator represents the reference seismic demand: γ_I is the importance factor assumed equal to 1, $a_{g,SLV} = 0.166$ g is the PGA corresponding to the return period of 475 years (Table 2) and $S = 1.25$ (slightly different from its assumption in Sect. 2.1 related to Post-NTC08 Guidelines). The seismic safety index automatically obtained by SIVARS for the “Pelella Palace” is $I_S = 0.506$.

Post–NTC08 Guidelines require two safety indexes for the SLV limit-state, i.e. one in terms of ground acceleration (factor of acceleration f_a) and the other in terms of return period (I_S^1). The spectral acceleration corresponding to the shear capacity of the building is defined as:

$$S_{SLV} = \frac{q \cdot F_{SLV}(\text{Post NTC08})}{e^* \cdot M} \tag{13}$$

where the mass participation factor $e^* = e_3^*$ can be calculated through the following relation, valid for a generic distribution of the masses and not constant storey heights:

$$e_3^* = \frac{\left(\sum_{j=1}^N m_j \phi_j\right)^2}{M \sum_{j=1}^N m_j \phi_j^2} \tag{14}$$

Analogously to parameter κ_i , when a triangular mode shape is assumed and the floor masses m_j are constant together with the storey heights, the expression of e_2^* of Eq. (11), only depending on the number N of floors, can be used.

With e_3^* and the other data reported in Table 3, it results $S_{SLV} = 2.219 \text{ m/s}^2$ for the “Pelella Palace”. The corresponding capacity in terms of seismic parameters according to Eq. (3), such as the return period (T_{SLV}), the ground acceleration capacity (a_{SLV}), F_0 and T_C^* , are computed by interpolation of the data in Table 2 related to the return periods of 72 and 101 years, assuming $S = 1.2$. The results are reported in Table 4.

Given the reference seismic action of the site for the SLV limit-state as characterized by $T_{R,SLV} = 475$ years, $a_{g,SLV} = 0.166 \text{ g}$ (Table 2), the index of seismic safety and the acceleration factor are respectively:

$$I_S^1 = \frac{T_{SLV}}{T_{R,SLV}} = 0.189$$

$$f_a = \frac{a_{SLV}}{a_{g,SLV}} = 0.484 \tag{15}$$

4 LV3 assessment level

The LV3 assessment level of the seismic safety of the “Pelella Palace” is performed through nonlinear static analysis by using the TREMURI computer program (Cattari et al. 2005; Lagomarsino et al. 2013). The three-dimensional model of the building is based on the identification of an equivalent frame consisting in vertical (piers) and horizontal (spandrels) macro-elements. The intersection areas between horizontal and vertical elements are modelled as rigid nodes. The 2D model of the façade wall and the 3D model of the entire building are sketched in Figs. 4 and 5, respectively.

Table 4 Seismic parameters relative to $S_{SLV}(T_f)$

| T_{SLV} (years) | a_{SLV} (g) | F_0 | T_C^* (s) | $S_{SLV}(T_f)$ (m/s^2) |
|-------------------|---------------|-------|-------------|-----------------------------------|
| 90 | 0.080 | 2.351 | 0.454 | 2.223 |

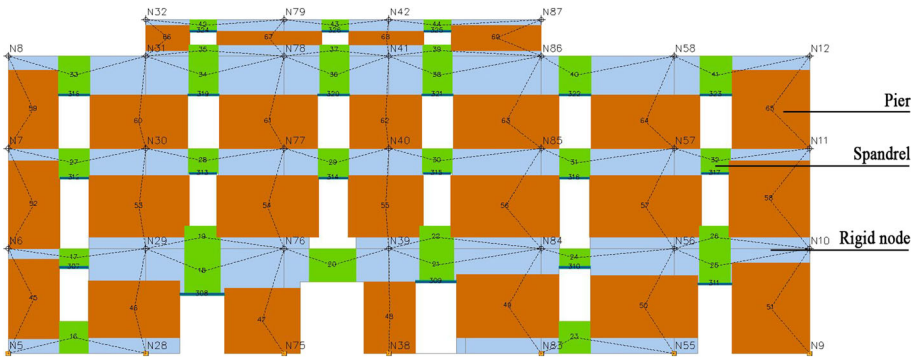


Fig. 4 Two-dimensional structural-element modelling of the façade wall

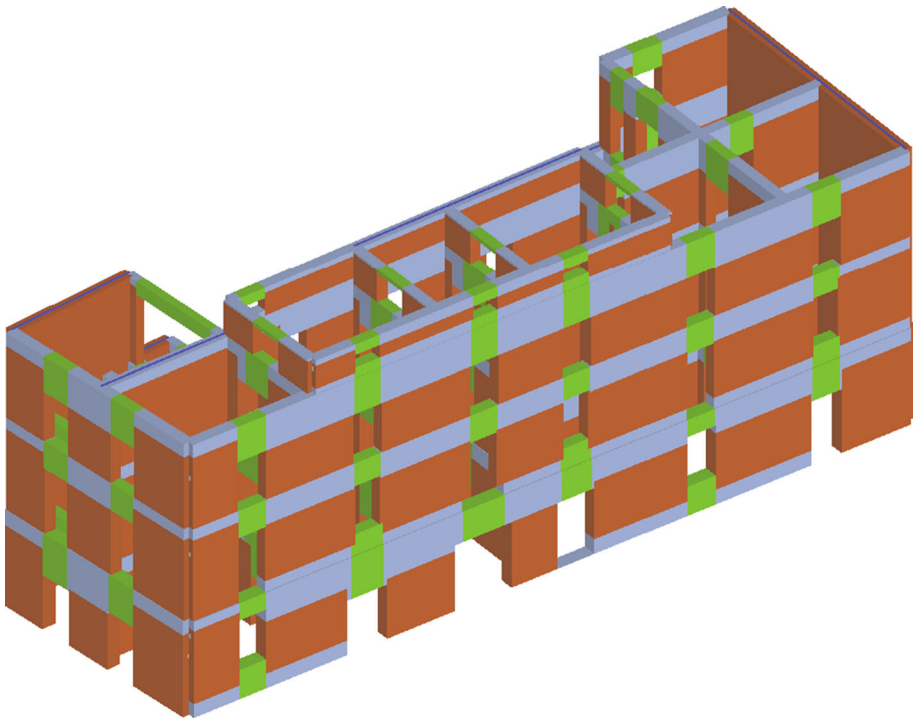


Fig. 5 Three-dimensional view of the structural-element model

The nonlinear behaviour of masonry piers and spandrels is assumed as elastic-perfectly plastic with initial cracked elastic stiffness (Calderini et al. 2009). The computation of the ultimate shear and bending strengths, based on the typical failure modes observed for masonry elements, follows the formulations provided by several seismic codes (NTC 2008; Eurocode 8 2005; ASCE/SEI 41/06 2007). In what concerns the shear behaviour, the failure modes of diagonal cracking and shear-sliding are considered as follows. The first one is described by the Turnšek and Čačovič criterion (1971) and is characterized by

diagonally oriented cracks that start at the centre of the element when the principal tensile stresses exceed a reference value. Shear-sliding failure is described by the Mohr–Coulomb criterion and is characterized by horizontal cracks with masonry units sliding upon the bed joints at one of the extremities of the element. Instead, concerning the flexural behaviour, the ultimate bending moment characterizing the rocking failure is calculated assuming a stress-block distribution of normal stresses at the compressed toe. Despite this classification, it is evident that also mixed modes are possible and quite common, while in this analysis, as a function of the current value of the axial force acting on the element, the minimum value of the ultimate strengths is assumed as the reference failure mode.

Relative to the plastic branch, the effects of cyclic actions are taken in account through the degradation of the stiffness (reduction of 50% as for LV1), while the ultimate drift of the generic masonry panel is defined through empirical formulations based on the prevailing failure mode, as reported by many international codes such as the Eurocode 8 (2005), the Italian NTC08 (2008), the American pre-standard FEMA 365 (2000). In particular, for the SLV of existing buildings, the Italian NTC08 simply assumes that the values of the ultimate drift are 0.6 and 0.4% of the height of the panel, corresponding to the bending and shear failure modes, respectively. When one of these ultimate drifts is reached by the masonry panel, it cannot withstand further loading increment and, as a consequence, a redistribution of the stresses takes place until the achievement of the ultimate displacement capacity d_u of the whole structure on the pushover curve (computed by considering the base shear decay equal to 20%).

The results presented in this section are related to a model with rigid floors. Afterwards a comparison is developed with a model which takes into account the deformability of the timber and steel floors in their plane. A preliminary modal analysis is performed to investigate the mass participation in each considered direction. From the results summarized in Table 5, it is evident that the first and third mode shapes are characterized by the greater amount of mass, contributing to about 68 and 76% of the total mass, respectively. Incidentally, it is worth noting that the fundamental period is very close to the approximated value of Eq. (2) used for LV1. In Figs. 6 and 7 the plan views of the first and third mode shapes at the third floor are reported, respectively.

The pushover analysis is conducted by considering two systems of horizontal forces applied at the storey levels and acting in the two orthogonal directions coinciding with the principal axes of the building:

- A system of forces proportional to masses (consistent with a soft ground storey response);
- A system of forces proportional to the first mode shape of the structure (it is able to represent the structural dynamic amplification).

These two distributions may be assumed as bounds for the seismic analyses of regular buildings: the actual result, coming from dynamic analyses, is usually assumed to be within these two solutions. In fact, since the pushover analysis is based on a nonlinear static procedure where the external loads monotonically increase up to failure, it neglects many significant aspects of the actual structural response, like the damage produced by reversal loads. The accuracy in the prediction of the static analyses may depend on the evolution of the structural response due to damage.

Such systems of static forces are applied according to 24 different possible loading conditions taking into account the variability of the verses of the forces and the accidental eccentricities of the mass centres (positive and negative), whose position at each level is considered as the origin of x and y axes (Figs. 6, 7). In particular, in order to counteract

Table 5 Modal analysis results considering the first ten mode shapes

| Mode shape | T (s) | m_x (kg) | $\%m_x$ | m_y (kg) | $\%m_y$ | m_z (kg) | $\%m_z$ |
|------------|---------|------------|---------|------------|---------|------------|---------|
| 1 | 0.40208 | 229 | 0.01 | 1346589 | 68.05 | 52 | 0.00 |
| 2 | 0.27961 | 204943 | 10.36 | 239776 | 12.12 | 85 | 0.00 |
| 3 | 0.26168 | 1505238 | 76.07 | 42189 | 2.13 | 339 | 0.02 |
| 4 | 0.16074 | 2528 | 0.13 | 134994 | 6.82 | 78 | 0.00 |
| 5 | 0.15406 | 292 | 0.01 | 118472 | 5.99 | 31 | 0.00 |
| 6 | 0.12529 | 295 | 0.01 | 9648 | 0.49 | 24 | 0.00 |
| 7 | 0.11014 | 58 | 0.00 | 24245 | 1.23 | 49 | 0.00 |
| 8 | 0.10322 | 117131 | 5.92 | 10213 | 0.52 | 4880 | 0.25 |
| 9 | 0.10290 | 27017 | 1.37 | 20194 | 1.02 | 420 | 0.02 |
| 10 | 0.09397 | 79349 | 4.01 | 5138 | 0.26 | 264 | 0.01 |

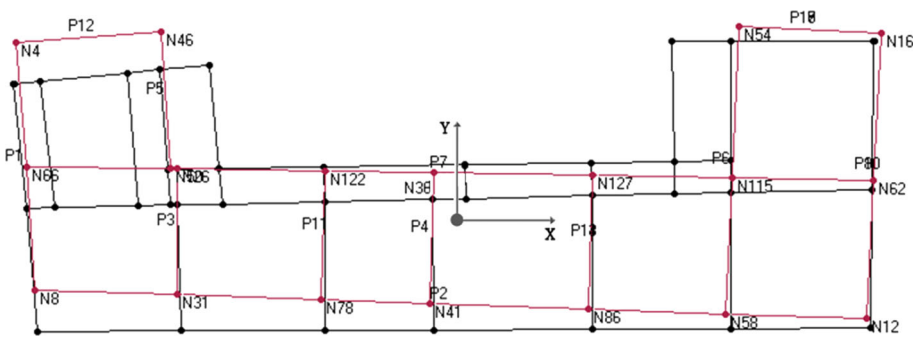


Fig. 6 Plan view of the first mode shape at the third floor

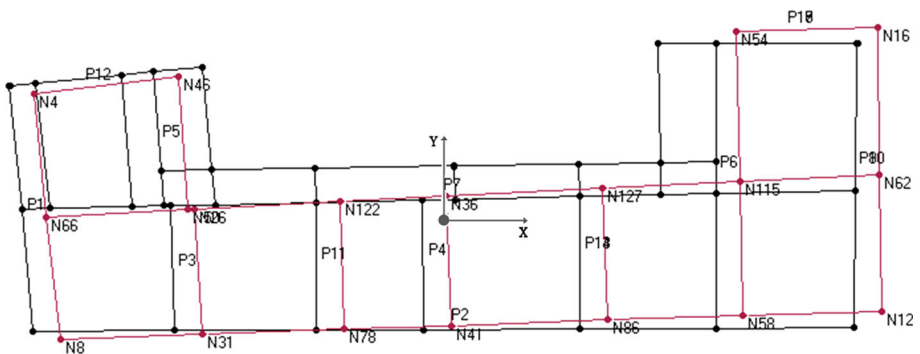


Fig. 7 Plan view of the third mode shape at the third floor

possible uncertainties on the position of the mass centres, the NTC08 (NTC 2008) prescribes to account for accidental eccentricities of the mass centres at each level equal to $\pm 5\%$ of the maximum length of the building in direction perpendicular to the seismic action.

Table 6 Results of the 24 loading conditions required by the pushover analysis

| Loading condition ID number | Direction | System of forces | Eccentricity (cm) | SLV | | | | |
|-----------------------------|-----------|------------------|-------------------|----------------|------------|-------|---------------|---------------|
| | | | | d_{max} (cm) | d_u (cm) | q^* | α_{u1} | α_{u2} |
| 1 | + x | Masses | 0.0 | 1.40 | 2.62 | 1.59 | 1.63 | 1.58 |
| 2 | + x | 1st Mode | 0.0 | 1.80 | 3.33 | 1.91 | 1.57 | 1.64 |
| 3 | - x | Masses | 0.0 | 1.37 | 3.20 | 1.36 | 1.92 | 1.66 |
| 4 | - x | 1st Mode | 0.0 | 1.69 | 4.25 | 1.59 | 1.88 | 1.66 |
| 5 | + y | Masses | 0.0 | 2.45 | 4.75 | 2.72 | 1.10 | 1.66 |
| 6 | + y | 1st Mode | 0.0 | 3.27 | 3.94 | 3.60 | 0.83 | 1.17 |
| 7 | - y | Masses | 0.0 | 2.37 | 2.24 | 2.30 | 0.95 | 0.94 |
| 8 | - y | 1st Mode | 0.0 | 2.83 | 3.51 | 2.66 | 1.13 | 1.19 |
| 9 | + x | Masses | 61.7 | 1.40 | 2.58 | 1.59 | 1.61 | 1.56 |
| 10 | + x | Masses | - 61.7 | 1.40 | 2.66 | 1.59 | 1.65 | 1.60 |
| 11 | + x | 1st Mode | 61.7 | 1.77 | 3.20 | 1.90 | 1.58 | 1.60 |
| 12 | + x | 1st Mode | - 61.7 | 1.78 | 3.29 | 1.91 | 1.57 | 1.64 |
| 13 | - x | Masses | 61.7 | 1.37 | 3.14 | 1.36 | 1.89 | 1.66 |
| 14 | - x | Masses | - 61.7 | 1.36 | 3.23 | 1.35 | 1.94 | 1.66 |
| 15 | - x | 1st Mode | 61.7 | 1.72 | 4.05 | 1.59 | 1.89 | 1.66 |
| 16 | - x | 1st Mode | - 61.7 | 1.81 | 4.23 | 1.57 | 1.91 | 1.66 |
| 17 | + y | Masses | 182.1 | 2.68 | 5.77 | 2.44 | 1.23 | 1.66 |
| 18 | + y | Masses | - 182.1 | 2.55 | 4.83 | 3.00 | 1.00 | 1.66 |
| 19 | + y | 1st Mode | 182.1 | 3.20 | 3.94 | 3.27 | 0.92 | 1.19 |
| 20 | + y | 1st Mode | - 182.1 | 3.31 | 4.18 | 3.97 | 0.76 | 1.22 |
| 21 | - y | Masses | 182.1 | 2.24 | 2.63 | 2.19 | 1.16 | 1.13 |
| 22 | - y | Masses | - 182.1 | 2.44 | 2.08 | 2.52 | 0.86 | 0.87 |
| 23 | - y | 1st Mode | 182.1 | 2.74 | 3.58 | 2.50 | 1.20 | 1.25 |
| 24 | - y | 1st Mode | - 182.1 | 2.94 | 3.51 | 2.90 | 1.04 | 1.16 |

The results of these analyses are reported in Table 6. For the SLV limit-state the reference parameters for the safety assessment are the displacement capacity and demand, respectively d_u and d_{max} , the ratio q^* between the shear force of the system, supposed indefinitely elastic, and the yielding strength of the equivalent nonlinear system. Since the safe condition requires $q^* < 3$ and obviously $d_u \geq d_{max}$, the loading conditions n. 6, 7, 18, 19, 20, 22 reported in Table 6 are not verified; they are all related to the y-direction that is confirmed the more vulnerable direction, as already outlined above for the first level assessment (LV1).

In the same table two values, say α_{u1} and α_{u2} , of the parameter α_u are also reported. This parameter α_u expresses the ratio between capacity and demand in terms of PGA, where the value of PGA corresponding to the capacity (PGA_C) is calculated on the basis of the nonlinear static procedures against the simplified linear static analysis (since it is based on the use of the q factor) used for the LV1 assessment. In particular, the two values α_{u1} and α_{u2} , derive from two different methods of determining the acceleration capacity.

Method 1, used to define α_{u1} , assumes for the capacity the same parameters of the seismic hazard (T_R, F_0, T_C) characterizing the reference demand for the SLV limit-state

(Table 2); only the peak ground acceleration is varied in order to match the value (PGA_C) corresponding to the displacement capacity d_u . The same goal of matching PGA_C with d_u is also pursued by the alternative method, say Method 2, by modifying all the seismic parameters (PGA, T_R, F_0, T_C). In this sense, the so obtained α_{u1} is analogous to the safety index I_S of the LV1 Pre-NTC08, while the index α_{u2} is analogous to the safety index f_a of the LV1 Post-NTC08. However, the minimum value between them can be considered for the sake of safety.

From Table 6 it is evident that the worst condition is given by the analysis n. 20 for y-direction ($\alpha_{u1} = 0.76$) and in Fig. 8 the pushover curve and the corresponding bilinear capacity curve related to this loading condition are represented. In this case, although the ratio capacity/demand in terms of displacement is $d_u/d_{max} = 1.26 > 1$, the verification is not satisfied because it also results $q^* = 3.97 > 3$.

Some considerations are also possible about the damage patterns exhibited by the walls. The most significant walls in y-direction, labelled as P1, P3, P4, P11 and P13, are shown in plan in Fig. 9, where Nj stands for Node j. Their damage patterns due to loading condition n. 20 are shown in Fig. 10.

In this figure it is evident that all the considered walls are characterized by the prevalence of bending collapse of the piers at the third floor. The spandrels, instead, result more vulnerable to shear failure. In particular the walls P1 and P3 show shear collapse of

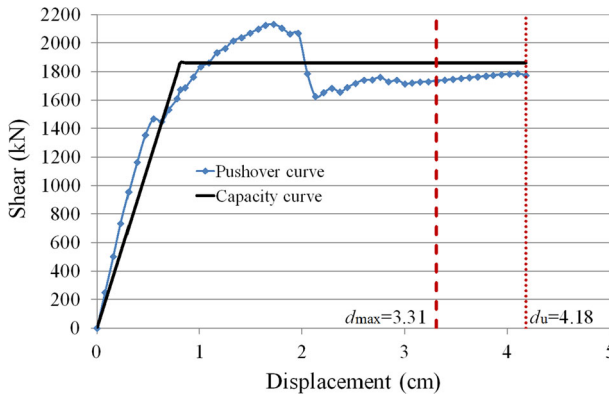


Fig. 8 Pushover and capacity curves related to the worst loading condition for y-direction

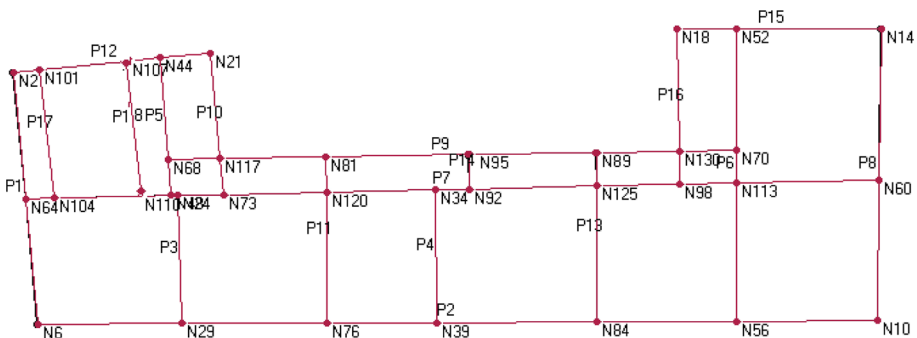


Fig. 9 Plan view of the ground floor with the numbering of the walls

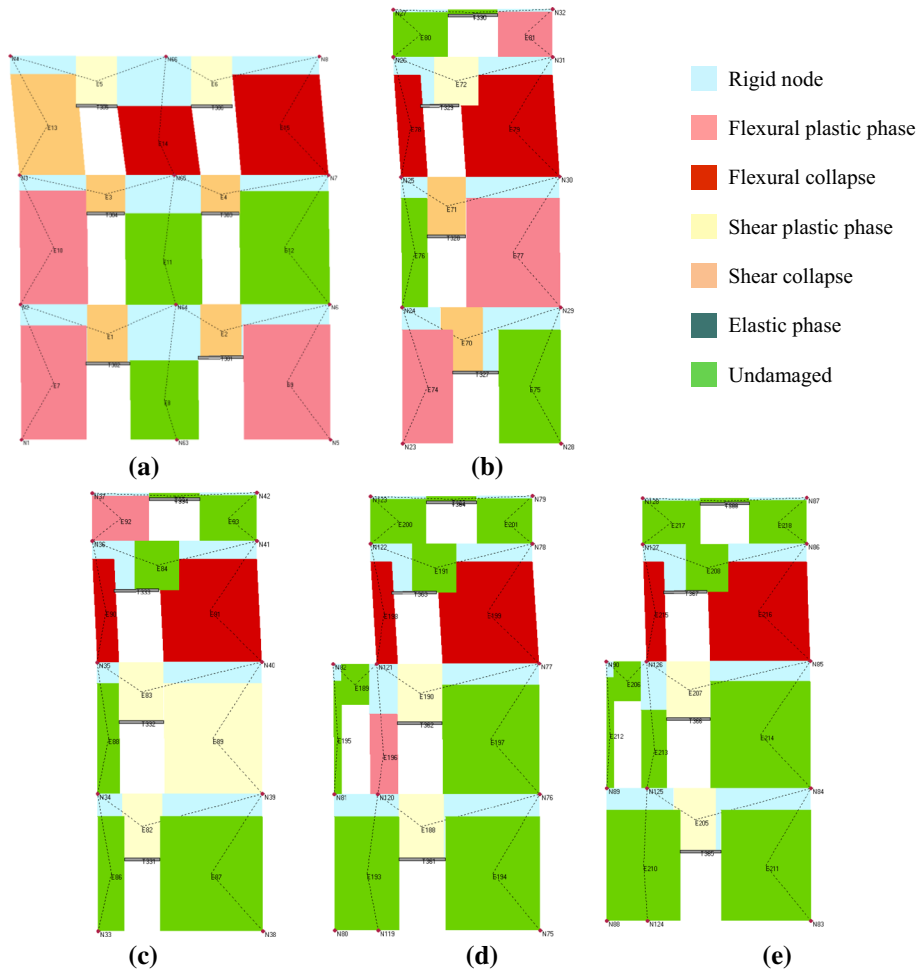


Fig. 10 Damage patterns related to the worst loading condition (n. 20) acting in y-direction. **a** P1 wall. **b** P3 wall. **c** P4 wall. **d** P11 wall. **e** P13 wall

the spandrels at the first and second floor, while the spandrels of the walls P4, P11 and P13 undergo the shear plastic phase. In general, also considering the other piers in flexural plastic phase, the walls P1 and P3 appear to be more vulnerable than the others. Since these are far from the centre of the masses, they can also be affected by greater displacements. On the other hand, it is worth noting that the failure modes of the spandrels are also influenced by the presence of lintels above the openings. These are modelled as timber elements with cross section area of 500 cm², elastic modulus of 11 kN/mm² and yielding strength of 20 MPa.

About the walls in x-direction, Fig. 11 shows the collapse mechanisms due to bending and shear for the piers at the first floor; bending plasticizations, instead, are prevalent for the piers at the third floor. The spandrels, as already noted for y-direction, are vulnerable to shear mechanisms. In particular, at the first and second floors they reach the collapse.

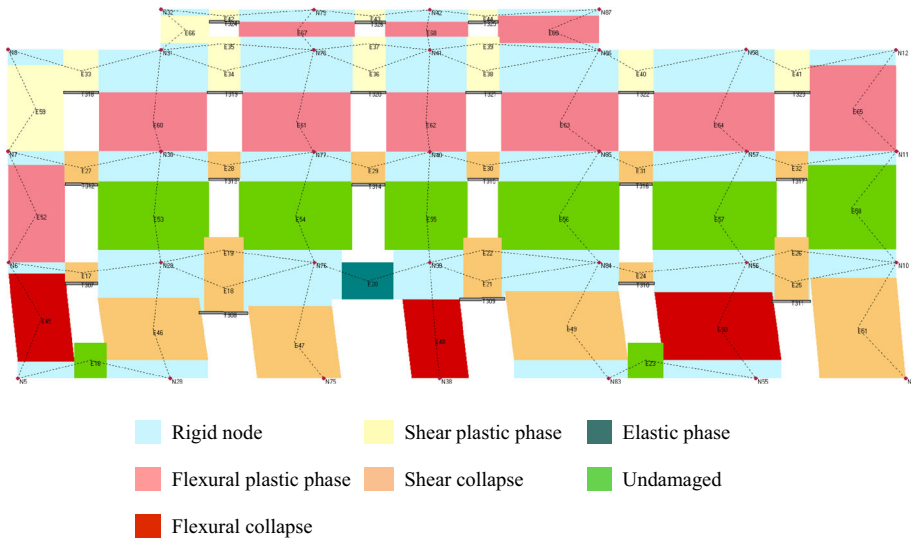


Fig. 11 Damage pattern of the P2 wall, related to the loading condition n. 3 acting in x -direction

4.1 Comparison between the models with rigid and flexible floors

The assumption made on the diaphragm stiffness may significantly affect the overall response of masonry buildings. In fact, in the limit case of “infinitely” flexible floors, there would be no load transfer from heavily damaged walls to still efficient structural elements. On the contrary, in the other limit case of floors assumed as “infinitely” stiff, this contribution could be overestimated.

As discussed in Galasco and Frumento (2011), the use of pushover analysis in case of masonry buildings with in-plane flexible diaphragms presents issues and difficulties which have not been yet taken into consideration in seismic codes. As an example, the choice of the control node is in such cases particularly critical, where a storey centre of mass cannot be assumed as the control node and significant differences can be observed in the displacements of different points at the same storey. Other interesting issues related to this topic are contained, among others, in the works of Magenes and Penna (2009) and Senaldi et al. (2014).

In this section, the deformability of the timber and steel floors in their plane is taken into account by using membrane finite elements in TREMURI program, according to the model proposed by Lagomarsino et al. (2013). These equivalent elements are characterized by a principal direction with two values of Young modulus along the two orthogonal directions (parallel, E_1 , and perpendicular, E_2 , to the spanning direction), Poisson ratio (ν) and in-plane the shear modulus (G_{eq}). The Young modulus E_1 represents the normal stiffness of the membrane along the principal direction (floor spanning orientation); it takes into account the degree of connection between the walls and the horizontal diaphragm and defines a link for the horizontal displacements of the involved nodes, also influencing the value of the axial forces transferred to the spandrels. The shear modulus G_{eq} , instead, represents the shear stiffness of the membrane and influences the repartition of the horizontal force among the walls. These quantities are evaluated from the stiffness of the

constitutive elements of the diaphragm, i.e. E_1 from the axial stiffness of the beams and G_{eq} from the shear stiffness of the slab.

The mechanical parameters that identify the equivalent stiffness of the two main typologies of unidirectional diaphragms for “Pelella Palace” are reported in Table 7.

The comparison between the two models is herein developed both in terms of acceleration and displacement capacity, the results of which are reported in Table 8. Although the worst loading conditions are still registered in y -direction (the corresponding ID numbers are represented in Table 5), it emerges that the modelling of flexible floors causes a decrease in the shear capacity of about 35% and in the safety index of about 41% with respect to the model with rigid floors. It is worth noting that α_{u1} corresponding to the minimum PGA_C is considered for the sake of the safety.

Moreover, all the 12 analyses with loading conditions in y -direction are not verified for the model with flexible floors, with the ratio capacity/demand in terms of displacement $d_u/d_{max} < 1$. On the contrary, this occurrence for the model with rigid floors is limited to two analyses only (n. 7 and 22 in Table 6), suggesting that in this case a more substantial redistribution of the horizontal actions between the piers allows limiting the local damage and improves the global response. It is also evident from Table 8 that the deformability of the horizontal structures affects the displacement capacity in much more substantial measure than the other parameters. In fact the percentage reduction of the displacement capacity is about 73%.

This result is also reflected into the plan deformations of the horizontal structures, as sketched in Fig. 12 for the third floor of the building. In fact, while a global response in terms of rigid rotation can be observed for the model with rigid floor (Fig. 12a), in the other model the plan deformation affects only the central portion of the “C” plan (Fig. 12b). In the latter case the “C” shape by itself behaves as a uniaxial spanning system between two “supports”, involving an “arch” deformation under horizontal loading, similar to the bending deformation of a simply supported beam. This effect could also be accentuated by the orientations of the horizontal diaphragms which are mixed within the “supports” and parallel to the direction of the seismic action (y -direction) in the central portion (Fig. 2). This means that the presence of flexible diaphragms does not allow redistribution of horizontal forces between walls, considering that these forces, proportional to tributary masses and the assumed mode shape, are applied to each node at the level of each floor.

Table 7 Mechanical parameters defining the equivalent stiffness of the diaphragms of “Pelella Palace”

| | E_1 (N/mm ²) | E_2 (N/mm ²) | G_{eq} (N/mm ²) |
|---------------|----------------------------|----------------------------|-------------------------------|
| Steel floors | 6825 | 0 | 62,900 |
| Timber floors | 10,000 | 0 | 10 |

Table 8 Comparison between flexible versus rigid floors models in y -direction

| Model | Loading conditions ID number | F_{SLNy} (corresp. to $\alpha_{u1,min}$) (kN) | $\alpha_{u1,min}$ | d_u (cm) |
|-----------------|------------------------------|--|-------------------|------------|
| Rigid floors | 20 | 1859.95 | 0.756 | 4.18 |
| Flexible floors | 6 | 1203.05 | 0.449 | 1.14 |
| Difference (%) | – | – 35% | – 41% | – 73% |

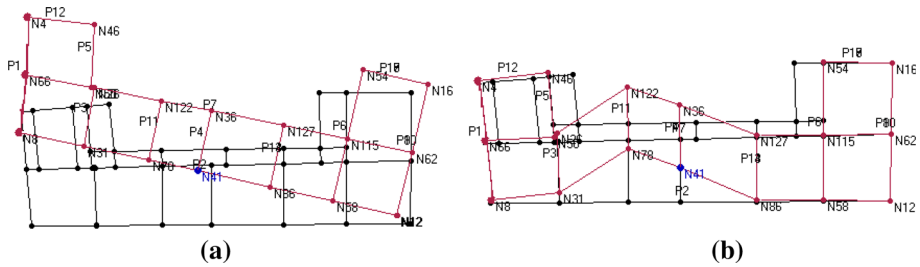


Fig. 12 Plan deformation at the third floor for **a** rigid and **b** flexible floors models, according to the worst conditions

However, it is worth underlining that the building is considered as an isolated structure, as specified in Sect. 2, otherwise the deformed shape shown in Fig. 12a could appear quite unrealistic. On the other hand, considering the results obtained for the building as representative of the seismic behaviour of a unit included in an aggregate would imply an overestimation of the displacement response, since the single structural unit would be subjected to a much higher demand. Also, the plan deformation in Fig. 12b could involve the possibility of out-of-plane failure mechanisms of the central walls in x -direction, but this aspect is not addressed here because it is beyond the scopes of this paper, as introduced above.

Notwithstanding, for both models, the ultimate condition is characterized by prevailing bending failure of the piers in y -direction at the third floor, as illustrated in Fig. 13. For the rigid model, however, the damage involves a greater number of walls, showing a better redistribution of the seismic action, as already noted.

5 Discussion of the Results

5.1 Comparison of LV1 results “Pre-NTC08” and “Post-NTC08”

In this section a critical comparison between the two versions of the Italian Guidelines on Cultural Heritage, Pre and Post-NTC08 (DCCM 2007, 2011), is carried out in order to highlight their pros and cons in view of results robustness.

As it is easy to verify from Table 3, both procedures provide similar values of the floor-shear resistances T_i with only minor differences due to the slight variability of the parameter ζ . Moreover, for all the storey levels the weaker direction is the y -direction, where the resistant area of the masonry piers is lower than that in x -direction. In particular, the top floor appears to be the weaker one, given the lowest value of the shear stress τ_{di} connected to the normal stress σ_{0i} and the reduction of the plane extension of this level, as already pointed out.

However, the two procedures strongly differ in the evaluation of the global shear capacity F_{SLV} , mainly because of the parameter κ that in the spirit of Post-NTC08 version makes the floor-shear resistances more comparable, putting them in relation with a global base shear. Thus, the global shear capacity according to Post-NTC08 Guidelines coincides with the base shear that activates the second floor shear resistance, i.e. F_{SLV} (Post-NTC08) = T_{y2}/κ_{y2} = 1220.19 kN. For the other floors, instead, it is expected that the assumed triangular shape mode provides floor-shear forces lower than the corresponding

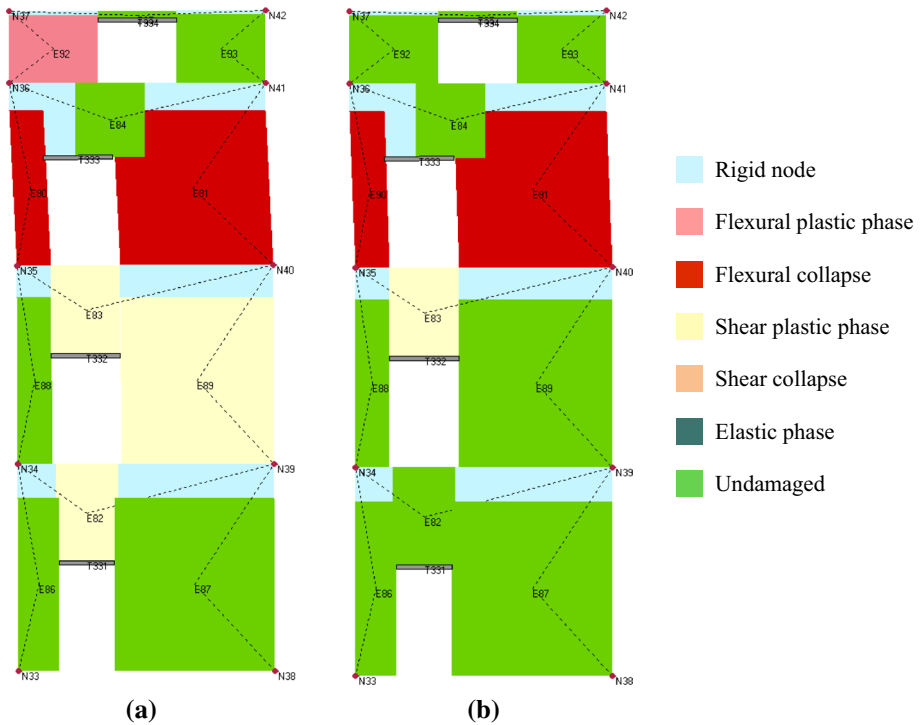


Fig. 13 Comparison of the damage patterns of the P4 wall, related to **a** rigid, **b** flexible floors models, resulting from the worst loading conditions

resistances calculated with Eq. (4). Thus, the value of F_{SLV} provided by Post-NTC08 version is about 2.96 times that provided by Pre-NTC08 Guidelines, i.e. $F_{SLV}(\text{Pre-NTC08}) = T_{y4} = 412.47 \text{ kN}$.

In order to test the validity of the results, an alternative option is explored by representing the fourth level as a mass affecting the third one. This allows considering a more regular distribution of the masses along the height and a better fitting of the validity conditions of the parameter κ provided by Eq. (9). Hence, the global shear capacity is evaluated with both expressions of κ provided by Eqs. (8) and (9), always assuming a triangular mode shape. The results reported in Table 9 show that if Eq. (8) is used to express κ_i , the global shear capacity and the identification of the more vulnerable floor are quite stable, as they are not influenced by the criterion adopted to represent the mass of the

Table 9 LV1 results assuming the fourth floor as an additional mass

| Floor | T_{yi} (Post-NTC08) (kN) | m_i (kg) | ϕ_i | κ_i (Eq. 8) | F_{SLVyi} (Eq. 8) (Post-NTC08) (kN) | κ_i (Eq. 9) | F_{SLVyi} (Eq. 9) (Post-NTC08) (kN) |
|-------|----------------------------|------------|----------|--------------------|---------------------------------------|--------------------|---------------------------------------|
| 1 | 1298.83 | 357,534 | 0.359 | 1 | 1298.83 | 1 | 1298.83 |
| 2 | 931.75 | 226,977 | 0.690 | 0.762 | 1222.68 | 0.9 | 1863.5 |
| 3 | 782.04 | 254,504 | 1 | 0.472 | 1657.77 | 0.7 | 4692.25 |

fourth level. In fact, when three levels are represented, the global shear capacity coincides with the base shear that activates the shear resistance of the second floor ($F_{SLV} = 1222.68$ kN, as shown in bold in Table 9), as it occurs by applying Eq. (8) to four levels (Table 3). And also the two values of the minimum F_{SLV} are very close to each other. On the other hand, if Eq. (9) is used, the global shear capacity is $F_{SLV} = 1298.83$ kN and the more vulnerable floor is identified with the ground floor. This means that the differences in the masses distribution and the storey heights are not so negligible. Lastly, it is important to highlight that if Eq. (9) is used considering four levels, results very different from the others are obtained (the global shear capacity coincides with the base shear that activates the shear resistance of the fourth floor). In sum, although the Post-NTC08 Guidelines have introduced a parameter to better relate the floor–shear resistances to a global base shear, this parameter is very sensitive to the regularity/irregularity of the masses distribution and the storey heights and instable results can be achieved in terms of the weakest floor even with small differences in such characteristics.

Another difference between the two versions is the computation of the mass participation factor e^* . As described above, for this parameter SIVARS system uses the expression e_1^* [the first one of Eq. (11)], assuming a storey mechanism for the weakest top floor; the Post-NTC08 Guidelines, instead, only consider the possibility of uniform collapse according to the cantilever idealization. Hence the simplified expression e_2^* [the second one of Eq. (11)] is suggested when a triangular mode shape is assumed and both the storey heights and the distribution of the masses are constant; a more generic expression e_3^* [Eq. (14)], instead, is proposed when all these conditions are not verified; however, it is worth noting from Table 3 that the value of e_2^* is quite similar to e_3^* (0.84 vs. 0.86), although referred to four levels (i.e. the case of more irregular distribution of the masses). It must be noted, on the other hand, that the hypothesis of uniform collapse is acceptable only when a significant amount of the participant mass is involved in the first modes of the building. Moreover, the exclusion of the storey mechanisms represents a critical issue of the Post-NTC08 Guidelines.

However, although these aspects could widely affect the seismic evaluation, the two obtained seismic safety indexes in terms of acceleration for “Pelella Palace” belong to the same order of magnitude and declare that the building is rather unsafe with reference to the SLV limit-state.

Finally, the most interesting innovation introduced by the new Guidelines is the criterion adopted to find the ground acceleration corresponding to the capacity of the building. In fact, Post-NTC08 Guidelines connect this ground acceleration to a return period that is not coincident with the return period of the reference seismic action expected for the SLV limit-state of the ordinary buildings [Eq. (1)]; obviously, different parameters of the seismic hazard and a different shape of the response spectrum are related to this capacity return period, $T_{R,SLV}$.

The main advantage of this approach is the possibility of associating to $T_{R,SLV}$ a value of the nominal lifetime consistent with the state of preservation of the construction. Hence, while for the new buildings this parameter is fixed at design stage, for the historical buildings the nominal lifetime can be defined on the basis of the return period $T_{R,SLV}$ corresponding to the building capacity by using Eq. (1). In fact, it will be $V_N = -T_{R,SLV} \times \ln(1 - P_{VR,SLV})/C_u$, where $P_{VR,SLV}$ and C_u are already defined in Sect. 2.1. For the “Pelella Palace” the value of V_N is about 7 years and it represents the period of validity of its seismic safety assessment. In this way, the results of the LV1 assessment can be expressed through a time parameter rather than a mechanical one and it can be managed in the context of the decisional processes of allocation of the economical resources.

5.2 Comparison of results provided by LV1 and LV3 assessment methods

The reliability and the limits of validity of the simplified mechanical model, characterized by a force-based approach, is critically appraised through its comparison with a more sophisticated model, which takes into account the nonlinear behaviour and the ductility reserve of the masonry building in the post-elastic range. It is worthy to recall that the two compared methods of seismic assessment are not alternative to each other, but belong to two different levels of evaluation. Moreover, this comparison should be significantly extended to a statistically sufficient number of buildings in order to get more general results and recommendations for researchers and practitioners.

In Table 10 the results of the seismic assessment methods LV1 and LV3 are reported, with reference to the weak y -direction. The LV3 results are related to the model with rigid floors as more reliable than those of the model with flexible diaphragms. In fact, despite the accuracy of the model adopted, there are still many uncertainties in the modelling criteria, the definition of the stiffness parameters and of the wall-to-diaphragm connections (Whitney and Agrawal 2015).

The value of the parameter f_a calculated by the LV1 method is about 64% of the corresponding minimum parameter $\alpha_{u,1}$ obtained by the LV3 [see Eq. (15) and Table 6]. Almost the same percentage is registered for the base shear. As expected, the LV1 approach appears to be more conservative than the LV3 method, because it assumes substantial simplifications for describing the structural behaviour. In fact, the seismic capacity of the building is measured by using mechanical-based parameters in place of an accurate structural model and, above all, in terms of forces rather than displacements, so that the strongly nonlinear behaviour of the structure is not properly considered.

About the comparison of the failure modes in y -direction, the LV3 provides a storey mechanism at the third floor as shown in Fig. 10, while the weakest floor according to the LV1 method is the second one. As explored in Sect. 5.1, also considering different options provided by Post-NTC08 for the determination of the parameter κ_i and representing the fourth floor as an additional vertical load related to the third floor, the results are different to each other for all the cases. This confirms that, despite the introduction of some parameters to improve the results, the LV1 method is still not adequate to capture the actual building behaviour in terms of failure mode. In fact, not only by its very nature it is unable to explore the dissipative capability of the whole structure, but it also excludes the possibility of the storey mechanism and, in addition, the possible flexural, shear and mixed failure modes of piers and spandrels cannot be represented by simple parameters to be assigned on the basis of the prediction of the global failure mode.

In conclusion, the LV1 method is capable to provide only a few information in agreement with the structural behaviour of the building since many aspects, such as the reference global models (cantilever and shear-type idealizations) and the failure modes of piers and spandrels (shear, bending and mixed modes), are still quite difficult to be

Table 10 Comparison between LV1 and LV3 analysis results

| | $\alpha_{u,1,\min}$ | f_a | F_{SLV_y} (kN) |
|-----|---------------------|-------|------------------|
| LV1 | – | 0.484 | 1220.19 |
| LV3 | 0.756 | – | 1859.94 |

represented by non-dimensional parameters. The results of the LV1 related to a single building are confirmed to be more meaningful in comparative terms with other similar buildings, in order to define a useful classification on the basis of their exposure to the seismic risk.

6 Conclusions

In this work the seismic response of a case study of a historical masonry building located in Afragola (Naples, Italy) was investigated by means of two different levels of assessment: the LV1 evaluation, based on simplified mechanical models according to the provisions of the Italian Guidelines on Cultural Heritage, and the LV3, based on the pushover analysis of the equivalent frame model. The seismic assessment of the case study building was carried out assuming as a reference the provision of the Italian Code. However, from the critical review of the obtained results more general findings arose. They concern in particular the relevance of a multi-level evaluation of the cultural heritage in defining the strategy of the interventions in the management of the seismic risk; the possibility to synthesize the outcomes of the evaluation through indexes related to mechanical parameters; the importance of such parameters in terms of the time validity of the safety assessment and their limits of validity in terms of failure modes; the influence of the hypotheses assumed about the deformability of the horizontal diaphragms in a 3D equivalent frame model.

As far as the first level of assessment (LV1) is concerned, the results obtained by the SIVARS system (Pre-NTC08) were compared with those obtained by the current version of the Guidelines, referred to NTC08. According to Pre-NTC08, the LV1 assessment was expressed in terms of a sole seismic safety index summarizing the comparison between seismic demand and capacity. Although Post-NTC08 is based on the same approach for the seismic assessment, it provides two safety indexes, one in terms of ground acceleration (factor of acceleration) directly comparable with the earlier index and the other in terms of return period, which adds useful information about the lifetime of the building. In fact, with reference to the return period of the seismic action corresponding to the building capacity, it is possible to define a more consistent and conservative value of the nominal lifetime, representing the validity period of the seismic safety assessment.

About the simplified mechanical models assumed by the two versions, a good agreement was reported with reference to the detection of the weaker direction along the axis of the shorter dimension in plan of the building and a prevailing failure mechanism of the masonry piers in this direction due to bending. Also, the two obtained safety indexes in terms of acceleration are of the same order of magnitude and denote that the masonry building under study is rather unsafe with reference to the achievement of the life-safety limit-state (SLV). However, some differences were found about the calculation of the base shear capacity and the corresponding ground acceleration and critical comments were provided on the main issues introduced by Post-NTC08 with respect to Pre-NTC08, in order to highlight their pros and cons. The sources of the main differences, such as the introduction of the two parameters ζ and κ (mainly the latter), the different computation of the mass participation, the limitation to the sole possibility of uniform collapse according to the cantilever idealization, were commented as well.

The reliability and the limits of validity of this simplified mechanical model, characterized by a force-based approach, was then critically appraised through its comparison with a more sophisticated model, which takes into account the nonlinear behaviour and the

ductility reserve of the masonry building in the post-elastic range. It is worthy to highlight that the two compared methods of seismic assessment, LV1 and LV3, are not alternative to each other, but belong to two different levels of evaluation and therefore more conservative results were actually expected for the former with respect to the latter evaluation method. However, some critical issues were addressed, mostly related to the damage patterns.

For the results of the LV3 analysis, developed by means of the TREMURI computer program for masonry structures, the weaker direction was identified along the axis of the shorter dimension in plan of the building and a prevailing failure mechanism of the masonry piers was observed in this direction due to bending. The safety indexes obtained by both approaches appear of the same order of magnitude, especially considering the deformability of the timber and steel floors in their plane. In fact, a further comparison between the models with rigid and flexible diaphragms was carried out within the LV3 pushover analysis to investigate the different capacity behaviour. The decrease of the safety index and the shear capacity of the building of about 41 and 35%, respectively, were obtained when flexible floors were assumed in the analysis, while the displacement capacity decreased of about 72%. Although there are still many uncertainties in the modelling criteria for flexible floors, this result does confirm that the stiffness of horizontal structures plays an important role on the global response of a masonry building.

In conclusion, the LV1 method is capable of providing only some information in agreement with the structural behaviour of the building since many aspects, such as the reference global models (cantilever and shear-type idealizations) and the failure modes of piers and spandrels (shear, bending and mixed modes) are still quite difficult to be represented by non-dimensional parameters. The results of the LV1 related to a single building are confirmed to be more meaningful in comparative terms with other similar buildings, in order to define a useful classification on the base of their exposure to the seismic risk.

The results provided by this study can easily be extended to any other masonry building in any earthquake-prone area, taking into account especially that the conventional parameters generally adopted in simplified mechanical models should be properly calibrated, even through parametric analysis, and particular attention should be devoted to the modelling of the horizontal diaphragms for accurate models.

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