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Comparison between a Simplified Approach and Pushover Analysis for a Case Study of Masonry Building in Naples

C. Casapulla [1], A. Maione[2], L.U. Argiento[3], A. Formisano[4], R. Landolfo[5]

Department of Structures for Engineering and Architecture, University of Naples Federico II, via Forno Vecchio 36, 80134, Naples, Italy, casacla@unina.it

Department of Structures for Engineering and Architecture, University of Naples Federico II, via Forno Vecchio 36, 80134, Naples, Italy, alessandramaione@tiscali.it

Department of Structures for Engineering and Architecture, University of Naples Federico II, via Forno Vecchio 36, 80134, Naples, Italy, archargiento1985@libero.it

Department of Structures for Engineering and Architecture, University of Naples Federico II, via Forno Vecchio 36, 80134, Naples, Italy, antoform@unina.it

Department of Structures for Engineering and Architecture, University of Naples Federico II, via Forno Vecchio 36, 80134, Naples, Italy, landolfo@unina.it

ABSTRACT: In this paper the seismic vulnerability assessment of a case study of masonry building located in Naples has been performed. First, a simplified analysis approach foreseen in the Italian Guidelines on Cultural Heritage, aligned with the Technical Codes for Constructions and based on a statistical evaluation of seismic behavior of masonry buildings, has been applied. Then, the achieved results have been compared with the more refined approach of nonlinear static analysis carried out by means of the TREMURI structural program. This is based on the schematization of masonry elements with the macro-element technique and the results are reported in terms of damage and collapse mechanisms of masonry walls, pushover curves and seismic safety index. The model with the additional assumption of all the floor diaphragms as completely rigid has also been analyzed and compared with the model with flexible horizontal structures. The comparison among results has allowed to have a clear indication of the seismic safety of the investigated building, useful to program future retrofitting interventions.

Keywords: Seismic vulnerability, Cultural Heritage, Pushover analysis, Equivalent frame model, Masonry collapse mechanisms

1 INTRODUCTION

The necessity of technical documents for the seismic risk assessment and seismic retrofitting of Cultural Heritage was dictated from the knowledge both that earthquake represents one of the main cause of damage to constructions and that many interventions carried out in the past have been resulted as ineffective or dangerous, the latter being often executed without considering the real behavior of the original structures.

The first guidelines having the purpose to evaluate the seismic behavior of monumental constructions were setup in Italy in 2005 after code OPCM 3431 (2005) was promulgated. In January 2010 a working group was created with the purpose to align the proposed guidelines with the Italian New Technical Codes for Constructions (NTC, 2008; MCIT 617, 2009). The implemented document was subjected to the inquiry of the Board of Public Works, which approved it on July 2010. In these new guidelines the performance based approach for safety evaluation was expressed in a very clear way, it being based on the definition of appropriate limit states (serviceability and ultimate), as well as on the corresponding reference actions to be assumed for checks. Moreover, a damage limit state for artistic heritage was added to guidelines as a further verification in order to check if the damages occurred into that patrimony are of a modest entity so that they can easily be restored without a significant lack of cultural value.

In recent years several researchers have concentrated interest on cultural heritage constructions which, not being subjected to a continuous maintenance, are affected by structural problems menacing buildings and people safety. A large part of this heritage is concentrated in European countries, where a valuable experience in conservation and restoration of masonry buildings (Giuffrè and Carocci, 1996; Lourenço et al., 2012) and churches (Lagomarsino et al., 2004; Binda et al., 2006) has been developed.

In particular, with reference to historical masonry buildings, the Italian Guidelines for the seismic risk evaluation and reduction of the Cultural Heritage (DCCM, 2011), aligned with the Italian New Technical Codes for Constructions (NTC, 2008; MCIT 617, 2009), give indications for three seismic analysis levels to assess their seismic safety and, consequently, to design retrofitting interventions: 1) LV1 level, used to provide the assessment at large scale; 2) LV2 level, used for evaluating local interventions on limited parts of buildings; 3) LV3 level, used to design interventions that influence the whole structural behavior or when an accurate global seismic response is required.

This approach was recently used by Formisano et al. (2013) and Indirli et al. (2013) to examine in a simplified way the seismic behavior of masonry building aggregates in San Pio delle Camere and Castelvecchio Subequo (districts of L'Aquila, Italy), subjected to the 2009 Abruzzo earthquake, and by Formisano (2013) to evaluate the seismic performance of a historical and monumental palace, including a little church, in Cento (district of Ferrara, Italy), which was subjected to the 2012 Emilia-Romagna earthquake.

Another case study is presented in this paper, according to the indications of the above codes and guidelines. The seismic behavior of Pelella Palace, a 19th century masonry building located in Naples (Italy), was examined with reference to LV1 and LV3 analysis levels. The influence of the assumptions made on the diaphragm stiffness was also investigated.

In particular, for the first evaluation level (LV1) the procedure of the informative system SIVARS, implemented on line by the MiBAC (Ministry of Cultural Heritage and Activities), was used as a reference (www.benitutelati.it). Besides, the approach of nonlinear static analysis was adopted for the evaluation level LV3 by using TREMURI computer program (Lagomarsino et al., 2013; Penna et al., 2013), based on the analysis of an equivalent three-dimensional frame with nonlinear behavior.

2 THE CASE STUDY: PELELLA PALACE

Pelella Palace is characterized by a "C" plan with a rectangular courtyard in the rear of the building, while the main entrance leads to open staircase serving the first and second floors. Placed in corner position of an urban block, the building is adjacent to a reinforced concrete building (Figs. 1, 2 and 3).

The vertical structures are made up of Neapolitan yellow tuff stones and traditional mortar, except for small portions of walls (corners of the atrium) consisting of solid bricks. The tuff wall typology is a three-leaf wall, with two outer shells and a thick inner core of rubble material.

In the absence of specific experimental data, a limited knowledge level LC1 was assumed for the purposes of the mechanical properties of the walls, corresponding to a confidence factor FC = 1.35. This factor was used to reduce the reference values for the mechanical strengths provided by the table C8A.2.1 of the MCIT 617 (2009).

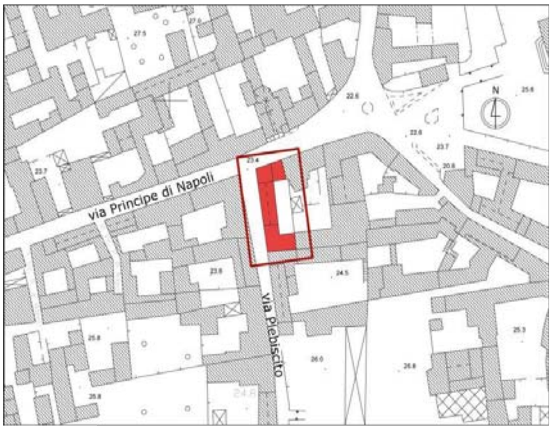

Figure 1. Location of Pelella Palace in the urban context.

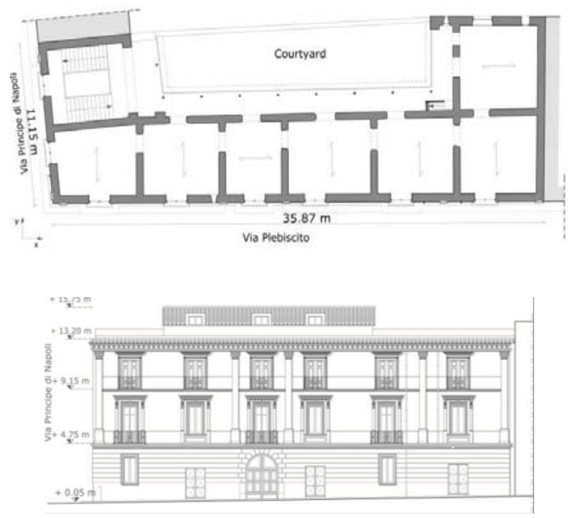

Figure 2. Plan and elevation layouts of Pelella Palace.

Figure 3. Front view of Pelella Palace.

However, some improved features of the masonry were taken into account, such as the presence of mortar of good quality and effective transverse connections between the outer leaves. With regard to the elastic modules, the average values derived from the aforementioned table C8A.2.1 was reduced to 50%, considering stiffness in cracked conditions. The assumed mechanical parameters of tuff masonry are reported in Table 1.

Table 1. Mechanical properties of tuff masonry.

E (N/mm^2)	G (N/mm^2)	f_m (N/mm^2)	τ_{0d} (N/mm^2)
1620	540	2.33	0.047

The horizontal structures are nearly always made up of timber floors, except for the presence of two barrel vaults on the ground floor (one with lunettes) and two diaphragms in the south-east area composed by steel beams and short-span brick vaults.

The original roof was made of wooden beams covered by a layer of tiles. At a later time, a second type of coverage was added to the original, made up of wooden beams and asbestos.

The staircase is a typical "open Neapolitan" typology, with sloping vaulted ceilings and dome cover.

Despite some phenomena of damage and decay, the building does not exhibit prejudices in its global behavior such as ground settlements or significant in-plane or out-of-plane failures of walls.

With regard to the definition of the seismic action, a soil type B (deposits of very dense sand, gravel, or very stiff clay), according to surveys made in the area, and the coefficient relative to topographic and stratigraphic conditions S = 1.2 were assumed.

2.1 LV1assessment level

The first assessment level (LV1) of the seismic safety of Pelella Palace was developed through the application of the simplified model proposed by the DCCM (2011) for the type "*palaces, villas and other structures with bearing walls and intermediate floors*". The assessment consists in the definition of a seismic safety index summarizing the comparison between seismic demand and capacity. This index is expressed in terms of ground acceleration (factor of acceleration) corresponding to the achievement of the Life Safety limit state (SLV) or in terms of return period.

The simplified model used to assess the seismic capacity is based on the assumption that the structure exhibits a global behavior with damage/collapse of the walls in their plan due to shear or bending.

The procedure first involves the calculation of the shear strength at a generic floor i, F_{SLVi}, assumed as coincident with the lowest value among those related to the two orthogonal x and y directions, which represent the main axes of the building. For the x-direction and the level i, for example, it assumes the following expression:

$$F_{SLVxi} = \frac{\mu_{xi} \cdot \xi_{xi} \cdot \zeta_{xi} \cdot A_{xi} \cdot \tau_{di}}{\beta_{xi} \cdot \kappa_i} \tag{1}$$

Equation (1) shows that the fundamental geometrical and mechanical parameters are the area A_{xi} of the resistant sections of masonry piers in the considered direction, the design value of the masonry shear strength τ_{di} , function of τ_{0d} in Table 1, and the average normal stress σ_{0i} agent at level i. Some coefficients are then introduced to bring into account, in a conventional way, the influence of other parameters, such as irregularities in plan (β_{xi}), uniformity of stiffness and strength of piers (μ_{xi}), resistance of spandrels (ζ_{xi}) and the prevailing type of failure in masonry walls (ξ_{xi}). The coefficient κ_i is finally introduced to relate the shear of a generic level i to the base shear.

The smallest value of the shear strength among those calculated at various levels, named F_{SLV}, is required to define the ordinate of the elastic response spectrum with reference to the SLV limit state:

$$S_{e,SLV} = \frac{q \cdot F_{SLV}}{e^* \cdot M} \tag{2}$$

where q is the behavior factor defined by NTC (2008) and M and e^* are the seismic mass and the participant mass fraction of the building, respectively.

The transition to the corresponding acceleration on horizontal rigid soil requires an iterative procedure to determine the return period T_{SLV} of the seismic action that involves the spectral acceleration given by Equation (2).

Table 2 summarizes the parameters needed for the LV1 evaluation level of Pelella Palace. These were calculated by the informative system SIVARS implemented by the Ministry of Cultural Heritage and Activities (MiBAC) and accessible through the web by institutional authorization (www.benitutelati.it). However, since the SIVARS system has not been aligned with the DCCM (2011), but to the previous DCCM (2007), the results have been corrected with the inclusion of the missing parameters κ_i and ζ_{xi} required by Equation (1) and the iterative procedure described above.

Table 2. Data for LV1 seismic assessment procedure.

M (kg)	T_1	e^*	q
1923182.83	0.4037 s	0.8384	3
I LEVEL			
τ_{dx} (N/m^2)	103 454.27	τ_{dy} (N/m^2)	103 454.27
A_x (m^2)	39.74	A_y (m^2)	30.65
μ_x	0.82	μ_y	0.80
ξ_x	1.00	ξ_y	0.80
ζ_x	1.00	ζ_y	0.80
β_x	1.03	β_y	1.25
κ_1	1.00	κ_1	1.00
F_{SLVx1} (kN)	3 272.64	F_{SLVy1} (kN)	1 298.83
II LEVEL			
τ_{dx} (N/m^2)	90 090.31	τ_{dy} (N/m^2)	98 123.70
A_x (m^2)	38.34	A_y (m^2)	25.25
μ_x	0.85	μ_y	0.80
ξ_x	1.00	ξ_y	0.80
ζ_x	1.00	ζ_y	0.80
β_x	1.15	β_y	1.25
κ_2	0.90	κ_2	0.90
F_{SLVx2} (kN)	2 836.52	F_{SLVy2} (kN)	1 127.55
III LEVEL			
τ_{dx} (N/m^2)	73 052.52	τ_{dy} (N/m^2)	73 052.52
A_x (m^2)	36.21	A_y (m^2)	25.09
μ_x	0.85	μ_y	0.80
ξ_x	1.00	ξ_y	0.80
ζ_x	1.00	ζ_y	0.80
β_x	1.25	β_y	1.20
κ_3	0.70	κ_3	0.70
F_{SLVx3} (kN)	2 569.44	F_{SLVy3} (kN)	1 117.23
IV LEVEL			
τ_{dx} (N/m^2)	56 608.33	τ_{dy} (N/m^2)	56 608.33
A_x (m^2)	15.92	A_y (m^2)	8.01
μ_x	0.80	μ_y	0.91
ξ_x	0.80	ξ_y	1.00
ζ_x	0.80	ζ_y	1.00
β_x	1.07	β_y	1.00
κ_4	0.40	κ_4	0.40
F_{SLVx4} (kN)	1 077.74	F_{SLVy4} (kN)	1 031.17

From the data in Table 2, it is evident that the global shear strength F_{SLV} coincides with the shear strength in the y-direction relative to the top level (minimum strength value); thus Equation (2) gives $S_{e,SLV} = 1.918$ m/s^2. This capacity acceleration corresponds to the return period $T_{SLV} = 66$ years and to the acceleration on horizontal rigid soil $a_{SLV} = 0.06927$g. Then, recalling that the reference seismic action of the site for the limit state SLV is characterized by $T_{R,SLV} = 475$ years and $a_{g,SLV} = 0.164$ g, the acceleration factor is:

$$f_a = \frac{a_{SLV}}{a_{g,SLV}} = 0{,}422 \quad (3)$$

Moreover, from Table 2 it appears that the weaker direction for all levels is the y-one, where the resistant area of masonry piers and the shear stress τ_d (depending on normal stress) are lower. It can also be noticed that the prevailing failure mechanism of piers in the y-direction is due to bending ($\xi_{yi} = 0.8$), with the exception of the top level ($\xi_{y4} = 1$). Instead, in the x-direction, the opposite phenomenon occurs, with prevailing shear failure mechanisms except for the top level. The prevailing type of mechanism is selected from the system SIVARS according to the piers slenderness and the loads applied on them.

2.2 LV3 assessment level

The LV3 assessment level of the seismic safety of Pelella Palace was performed through nonlinear static analysis by using the TREMURI computer program (Lagomarsino et al., 2013; Penna 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) macroelements. The intersection areas between horizontal and vertical elements are modeled as rigid nodes. The nonlinear behavior of masonry piers is assumed as elastic-perfectly plastic with initial cracked elastic stiffness; the strength criteria depend on the possible failure modes, i.e.: flexure-rocking, sliding shear and shear-diagonal cracking. The formulation is consistent with the recommendations included in several seismic codes (NTC, 2008; Eurocode 8, 2005; ASCE/SEI 41/06, 2007), since strength criteria defined for both bending and shear failure modes can be easily implemented and adopted to define the lateral strength of the different structural elements.

Relatively to the plastic branch, the effects of cyclic actions are taken in account through the degradation of the stiffness, while the ultimate limit state in terms of displacement is based on the failure of the generic panel through the maximum drift (d_u), which depends on the prevailing failure mode occurred in the panel. For existing buildings, the SLV values of the ultimate drift are assumed to be 0.6% and 0.4% of the inter-storey height, corresponding to the bending and shear failure modes, respectively.

Regarding the floor elements, the computer program allows to take into account the deformability in their plane, through modeling membrane finite elements with equivalent stiffness properties. Two types of floors characterize the building: floors with steel beams and hollow blocks and floors with wooden beams and planks. Even for the existing vaults, it is possible to define an equivalent horizontal stiffness as a function of their type, their thickness, the characteristics of materials and the type of connection to walls.

The pushover analysis was conducted considering two systems of horizontal forces applied at the level of floors and acted in the two orthogonal directions coinciding with the principal axes of the building:

a) a system of forces proportional to masses;

b) a system of forces proportional to the first vibration mode.

Such systems of static forces were applied according to 24 different possible load conditions to take account of the variability of the verses and of the accidental eccentricities of the mass center. Table 3 shows the results related to the worst load conditions for the x and y-directions. The reference parameters for the verification are the capacity and demand displacements, respectively d_u and d_{max}, while q^* is the ratio between the shear force of the system, supposed indefinitely elastic, and the yielding strength of the equivalent nonlinear system (with the limitation $q^* < 3$); the parameter α_u has the same meaning of f_a, being the ratio between capacity/demand in terms of PGA.

Table 3. Results related to the worst load conditions deriving from LV3 analysis method.

Dir.	Load cond.	Ecc. (cm)	d_u (cm)	d_{max} (cm)	q^*	α_u
+Y	1st mode	0	1.14	2.63	5.64	0.449
-X	1st mode	-61.7	2.91	2.05	1.887	1.351

The results of LV3 analysis confirm that y is the weak direction, also when the nonlinear behavior and the displacement capacity are taken into account. Figure 4 shows the storey mechanism at the third level, where flexural failure is prevailing for the masonry piers.

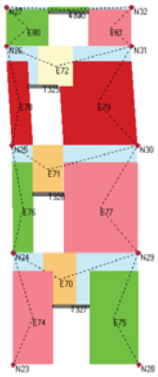

Figure 4. Flexural failure mode at the third floor according to LV3 analysis level.

2.3 Comparison of results provided by LV1 and LV3 assessment methods

The results of the seismic assessment methods LV1 and LV3 are compared and summarized in Table 4, with reference to the weak y-direction.

Table 4. Comparison between LV1 and LV3 analysis results.

	α_u	f_a	F_{SLVy} (kN)
LV1	-	0.422	1 031.17
LV3	0.449	-	1 163.50

Regarding the capacity/demand relationship in terms of PGA corresponding to the achievement of SLV, it results that the value of this parameter (f_a) calculated by the LV1 method is approximately the same than that (α_u) obtained from the application of LV3 one. Also, the base shear according to LV1 is slightly lower (of about 11%) than that calculated with the global response analysis LV3. Therefore, the LV1 approach appears to be more conservative than the other method, because it assumes substantial simplifications for describing the structural behavior: the seismic capacity of the building, in fact, is measured in terms of forces rather than displacements, so that the strongly nonlinear behavior of the structure is not properly considered. Actually, the same order of magnitude of the two parameters is mainly due to the fact that the presence of flexible diaphragms in the palace strongly reduces its seismic response with respect to the condition of infinitely stiff floors, generally assumed for LV3 analysis. This aspect is investigated in detail in the next section. In addition, it must be noted that the safety parameter is more meaningful in terms of risk classification than in terms of structural response characterization.

About the comparison of failure modes in the *y*-direction, LV3 provides a storey mechanism at the third level (Fig. 4), while the weaker level according to LV1 method is the fourth one. By both methods, however, the prevailing failure mode for the masonry piers in the *y*-direction is of flexural type.

2.4 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 (Lagomarsino et al., 2013).

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.

Within TREMURI software, the flexible timber floors in Pelella Palace are first modeled as membrane finite elements with equivalent stiffness properties (Table 3) and then a new model assuming the floor behavior as completely rigid is analyzed (Table 5).

Table 5. Results related to the worst load conditions of LV3 analysis of the model with rigid floors.

Dir.	Load cond.	Ecc. (cm)	d_u (cm)	d_{max} (cm)	q^*	α_u
+Y	1st mode	-182.1	4.18	3.31	3.968	0.756
+X	1st mode	-61.7	3.29	1.78	1.913	1.568

The comparison between the two models is developed both in terms of capacity and global verification.

With regard to the assessment of the overall capacity, the results corresponding to the two models in the weaker *y*-direction are compared in Table 6. By this comparison it emerges that the rigid floors allow an increase in shear capacity of about 54.6% and in terms of acceleration safety parameter (α_u) of about 68.4%.

Table 6. Comparison between flexible vs. rigid floors models in the *y*-direction.

Model	F_{SLVy} (corresp. to $\alpha_{u,min}$)	$\alpha_{u,min}$	d_u (cm)
Flexible floors	1 203.05 kN	0.449	1.14
Rigid floors	1 859.95 kN	0.756	4.18
Difference (%)	54.6	68.4	267

With regard to the results of global verification, all the 12 analyses with load conditions in the *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, for the model with rigid floors, this occurrence is limited to two analyses only. Thus, also the displacement capacity increases with the stiffness of the horizontal structures and this increment is much more substantial (about 3.7 times, corresponding to 267%) than that of the other parameters.

About the global response, a meaningful difference is evident from the comparison between the horizontal deformations of the plan at the third level of the two models (Fig. 5). It must be noted that floors are oriented along the *y*-direction for the central portion of the building and along the *x*-direction near the corners.

Notwithstanding, for both models the ultimate condition is characterized by prevailing bending failure of the piers in the *y*-direction at the third level, as illustrated in Figure 4. For the rigid model, however, the damage involves a greater number of walls, showing a better redistribution of the seismic action.

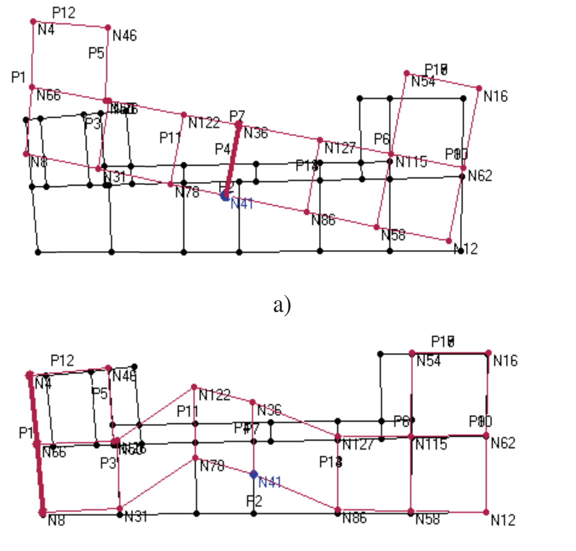

Figure 5. Plan deformation at the third level for a) rigid and b) flexible floors models.

3 CONCLUSIONS

In this work the seismic vulnerability of a case study of historical masonry building located in Naples has been investigated by means of two different approaches based on the Italian Guidelines on Cultural Heritage: the LV1 analysis approach and the LV3 one, the latter being executed with the TREMURI calculation program for masonry structures.

Both these approaches have highlighted a 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. By comparison it emerges that the LV1 assessment level provides more conservative results than LV3 one, because the former underestimates the structural ductility and assumes substantial simplifications for describing the structural behavior. However, the safety indexes obtained by both approaches appear of the same order of magnitude due to the presence of flexible diaphragms (timber floors).

A further comparison between the models with flexible and rigid diaphragms has been carried out within the LV3 pushover analysis through the TREMURI computer program. The increasing of the safety index and the shear capacity of the building of about 68.4% and 54.6%, respectively, are obtained when rigid floors are assumed in the analysis, together with the displacement capacity increased of about 3.7 times. This confirms that the stiffness of horizontal structures plays an important role on the global response of a masonry building.

In the whole, the seismic vulnerability analysis carried out in this paper by means of LV1 and LV3 approaches has provided a clear picture of the building deficiencies, useful to program future retrofitting interventions.

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