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A SIMPLIFIED APPROACH FOR THE SEISMIC VULNERABILITY ASSESSMENT: APPLICATION TO A MASONRY BUILDING IN NAPLES

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Abstract. *The seismic vulnerability assessment of a case study of masonry building located in Naples was performed by means of the first level of assessment (LV1) provided by the Italian Guidelines on Cultural Heritage. According to a large scale evaluation, this level is based on a simplified model of the global seismic behavior of masonry buildings. The results obtained by using the earlier version of the guidelines, implemented on line by the Ministry of Cultural Heritage and Activities (MiBAC) are expressed in terms of a sole seismic safety index summarizing the comparison between seismic demand and capacity. Instead, the updated version of the rules provides two safety indices, 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. The results of the two versions were largely discussed and compared and a good agreement was revealed with reference to the detection of the weaker direction and the prevailing failure mechanism. However, some differences were found about the calculation of the base shear capacity and the corresponding ground acceleration.*

1 INTRODUCTION

Often existing masonry buildings are the result of constructions, changes and modifications that have been developed over centuries. When located in earthquake-prone areas these are also exposed to seismic events which still represent one of the main causes of damage to constructions. In many cases, 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 behavior.

In recent years, several researchers have concentrated interest on cultural heritage constructions spread across European countries [1-8], while the capacity methods of analysis have been progressively abandoned in favor of performance-based techniques [9]. In this context it is worth mentioning the work of Lagomarsino [10] presenting a displacement-based procedure for rocking masonry structures based on the incremental limit analysis of a nonlinear equivalent SDOF system and on an adapted capacity spectrum method.

The theme of assessment and reduction of seismic risk of historical constructions is becoming more and more relevant in Italy, due to the huge number of potentially vulnerable heritage structures, and a number of Codes and guidelines have been produced. Specifically it's worthy of consideration:

• OPM n. 3274 (2003), updated with OPM. n. 3431 (2005) [11];

• Technical Rules for Constructions, proclaimed in 2008 (shortly named NTC 2008) [12];

• Instructions for the application of the New Technical Rules for Constructions (shortly named Circular n.617/2009) [13];

• Guidelines about the preservation of historical and architectonical heritage: "Seismic risk evaluation and reduction of the cultural heritage" (shortly named DCCM 2007) [14] and "Seismic risk evaluation and reduction of the cultural heritage, in alignment with the Technical Rules for Constructions" (shortly named DCCM 2011) [15].

The whole of these standards and rules, more advanced than the Eurocodes dealing with these arguments (specifically Eurocode n. 8 [16]), is a legacy of knowledge deriving from the Italian seismic recent experiences (just like the Umbrian earthquake of 1997, the Molise earthquake of 2002 and the Abruzzo earthquake of 2009).

While the performance-based techniques are being developed particularly for the out-of-plane seismic behavior of masonry structures in static and dynamic fields, also by means of original strategies [17, 18], the Italian Guidelines for the seismic risk evaluation and reduction of the cultural heritage [15], aligned with NTC 2008 [12, 13], 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.

Whilst extensive literature is available for LV2 and LV3 level assessments [19-24], yet little attention has been focused on LV1. The case study of Pelella Palace, a 19th century masonry building located in Naples (Italy), is described in this paper with reference to LV1. In particular, the procedure of the informative system SIVARS, implemented on line by MiBAC (Ministry of Cultural Heritage and Activities), is used as a reference [25] and the results are compared with those calculated with the improved procedure aligned with the NTC 2008. In [26] these results were also compared with those obtained by nonlinear pushover analysis.

2 THE CASE STUDY: PELELLA PALACE

Pelella Palace is a 19^{th} century masonry building located in Naples (Italy). It 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 Circular n. 617/2009 [13].

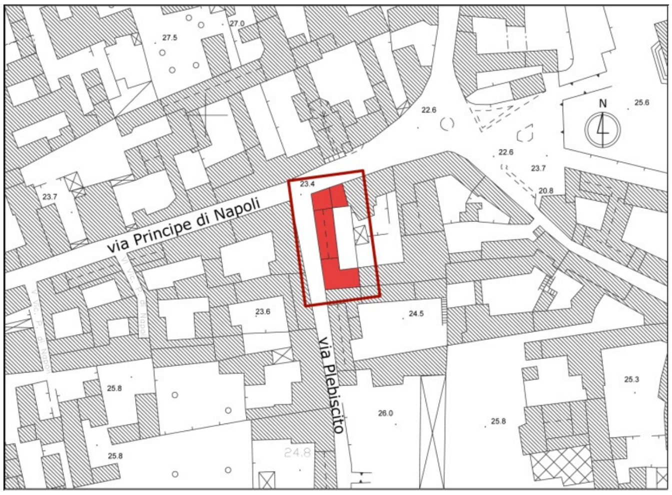

Figure 1: Location of Pelella Palace in the urban context.

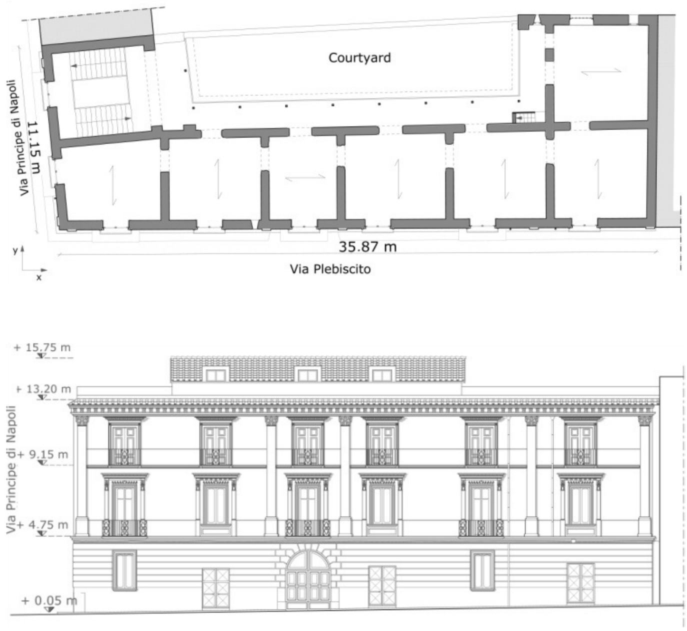

Figure 2: Plan and elevation layouts of Pelella Palace.

Figure 3: External 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. The importance of the good quality of masonry and of the connections between elements o the building was highlighted, among others, by Borri and De Maria [7].

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.

2.1 Reference seismic actions

In order to assess the seismic demand it is necessary 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. Taking a nominal lifetime very long, which in theory would be required for the cultural value of the artifact, would result in the definition of severe reference seismic actions and therefore 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 was assumed the nominal lifetime V_N = 50 years and the coefficient of use C_u = 1, so defining a reference lifetime $V_R = C_u \times V_N$ = 50 years.

The alignment of Directive 2011 [15] to the technical standards NTC 2008 [12, 13] has meant that, with reference to the cultural heritage, the probability of exceedance accepted for the seismic action is the same as for ordinary buildings. In particular for the SLV (the limit state of preservation of life), the probability of surplus accepted is equal to 10%. This parameter, together with V_R leads to define a reference period $T_{R,SLV}$ = 475 years. The parameters of the seismic hazard corresponding to such a reference period, according to NTC 2008, are (Table 1): $a_{g,SLV}$ = 0.164 g, F_0 = 2.39, T_{C^*} = 0.35 s.

Moreover, a soil type B (deposits of very dense sand, gravel, or very stiff clay) was assumed according to surveys made in the area, along with a coefficient S = 1.2, relative to topographic and stratigraphic conditions. The fundamental period T_1 of the building under study was evaluated as 0.4037 s, through the simplified formula of NTC 2008 (total height = 16.2 m).

Other return periods and corresponding parameters of seismic hazard provided by NTC 2008 are recorded in Table 2. The last column shows the ordinates of the elastic response spectrum relative to T_1 according to the formula [12]:

$$T_B \le T_1 < T_C \qquad S_{e,SLV}(T_1) = a_{g,SLV} \cdot S \cdot F_0 \tag{1}$$

where T_B and T_C are some reference parameters of the elastic spectrum.

Table 2: Seismic hazard parameters and spectral accelerations for SLV limit state, relative to soil type B and different return periods (NTC 2008).

$T_{R,SLV}$ [years]	$a_{g,SLV}$/g	F_0	T_C* [s]	T_C [s]	S	$S_{e,SLV}(T_1)$ [m/s^2]
30	0.046	2.339	0.286	0.404	1.2	1.297
50	0.060	2.35	0.314	0.435	1.2	1.693
72	0.072	2.345	0.326	0.449	1.2	2.039
101	0.085	2.354	0.333	0.456	1.2	2.410
140	0.100	2.352	0.338	0.462	1.2	2.809
201	0.117	2.346	0.343	0.467	1.2	3.308
475	0.164	2.389	0.350	0.475	1.2	4.715
975	0.209	2.460	0.354	0.479	1.195	6.133
2475	0.274	2.574	0.356	0.482	1.117	7.891

2.1 LV1assessment level

The assessment level (LV1) of the seismic safety of Pelella Palace was developed through the application of the simplified model proposed by the DCCM 2011 [15] 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 SLV limit state 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 strengths at a generic floor i, F_{SLVi}, along two orthogonal x and y-directions (main axes of the building). For the x-direction and level i, for example, the following expression is assumed:

$$F_{\mathrm{SLV}xi} = \frac{\mu_{xi} \cdot \xi_{xi} \cdot \zeta_{xi} \cdot A_{xi} \cdot \tau_{di}}{\beta_{xi} \cdot \kappa_i} \quad (2)$$

Eq. (2) shows 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}, 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 strength at a generic level *i* to the base shear.

In Table 3 the shear strength for each level and direction of Pelella Palace are reported. The column "Pre NTC 2008" refers to the results obtained by the informative system SIVARS implemented by the Ministry of Cultural Heritage and Activities (MiBAC) and accessible through the web by institutional authorization [25]. Since the SIVARS system is consistent with the DCCM 2007 [14] which was not yet aligned with NTC 2008, the second column "Post NTC 2008" reports the updated results with the inclusion of the missing parameters κ_i and ζ_i required by Eq. (2). Regarding the parameter ζ_i, which takes account of the resistance of the spandrels, it is assumed the value 1 (resistant spandrels) in cases where the SIVARS system provides $\xi_i = 1$ (shear failure of the piers) or value 0.8 (weaker spandrels) in cases where it is $\xi_i = 0.8$ (bending failure of the piers).

Table 3: Shear strengths at various levels and directions of Pelella Palace.

M [kg]	T_1 [s]	*q*
1923182.83	0.4037 s	*3*

I LEVEL	Pre NTC 2008 (SIVARS)	Post NTC 2008 (Eq. (2))
$F_{\mathrm{SLV}x1}$ [kN]	3272.64	3272.64
$F_{\mathrm{SLV}y1}$ [kN]	1623.54	1298.83
II LEVEL	Pre NTC 2008 (SIVARS)	Post NTC 2008 (Eq. (2))
$F_{\mathrm{SLV}x2}$ [kN]	2552.87	2836.52
$F_{\mathrm{SLV}y2}$ [kN]	1268.49	1127.55
III LEVEL	Pre NTC 2008 (SIVARS)	Post NTC 2008 (Eq. (2))
$F_{\mathrm{SLV}x3}$ [kN]	1798.61	2569.44
$F_{\mathrm{SLV}y3}$ [kN]	977.58	1117.23
IV LEVEL	Pre NTC 2008 (SIVARS)	Post NTC 2008 (Eq. (2))
$F_{\mathrm{SLV}x4}$ [kN]	538.87	1077.74
$F_{\mathrm{SLV}y4}$ [kN]	412.47	1031.17

The smallest value of the shear strength among those calculated at various levels represents the shear strength of the building which allows to define the spectral acceleration capacity with reference to the SLV limit state. From the data in Table 3, it is evident that the global shear strength for Pelella Palace is controlled by the shear strength in the *y*-direction relative to the top level, for which, however, different values can be obtained for Pre and Post NTC 2008. Then, according to the old and new mentioned guidelines, these values are used in two different ways to define the ground acceleration corresponding to the capacity of the building.

In fact, before the alignment of the rules, this acceleration named a_{SLV}, which takes into account the features of the soil, was evaluated through the relation between the base shear of the building (assumed coincident with the shear capacity F_{SLV}) and its spectral acceleration:

$$a_{\mathrm{SLV}} = \frac{q \cdot F_{\mathrm{SLV}}}{e^{*} \cdot M \cdot C(T)} \tag{3}$$

where q is the behavior factor, M and e^{*} are the seismic mass and the participant mass fraction of the building, respectively, and C(T) is the coefficient correspondent to F_0 in the new guidelines.

So, according to the old guidelines a sole safety index was identified as:

$$I_{\mathrm{S}} = \frac{a_{\mathrm{SLV}}}{\gamma_I \cdot S \cdot a_{g,SLV}} \tag{4}$$

where the denominator represents the reference seismic demand (γ_{I} is the importance factor). The seismic safety index automatically obtained by SIVARS for Pelella Palace was $I_{\mathrm{S}} = 0.495$.

On the other hand, the guidelines aligned with NTC 2008 require some development to define two safety indexes, one in terms of ground acceleration (factor of acceleration) corresponding to the achievement of the SLV limit state and the other in terms of return periods. In fact, the spectral acceleration corresponding to the shear capacity of the building is defined as:

$$S_{\mathrm{SLV}} = \frac{q \cdot F_{\mathrm{SLV}}}{e^{*} \cdot M} \tag{5}$$

For Pelella Palace Eq. (5) gives $S_{\mathrm{SLV}} = 1.918$ m/s^2. The corresponding seismic parameters according to Eq. (1), such as the acceleration on horizontal rigid soil (a_{SLV}), F_0 and T_{C}^{*} and the return period (T_{SLV}), are computed by interpolation of the data in Table 2. The results are reported in Table 4.

Table 4: Seismic parameters relative to $S_{\mathrm{SLV}}(T_1)$.

T_{SLV} [years]	a_{SLV}/g	F_0	T_{C}^{*} [s]	T_{C} [s]	S	$S_{\mathrm{SLV}}(T_1)$ [m/s^2]
66	0.069	2.346	0.323	0.445	1.2	1.918

Thus, remembering that the reference seismic action of the site for the limit state SLV is characterized by $T_{\mathrm{R,SLV}} = 475$ years and $a_{\mathrm{g,\,SLV}} = 0.164$ g (Table 2), it follows that the indices of seismic safety and the acceleration factor are respectively:

$$\begin{aligned} I_{\mathrm{S}} &= \frac{T_{\mathrm{SLV}}}{T_{\mathrm{R,SLV}}} = 0.14 \\ f_{\mathrm{a}} &= \frac{a_{\mathrm{SLV}}}{a_{\mathrm{g,SLV}}} = 0.42 \end{aligned} \tag{6}$$

3 DISCUSSION OF THE RESULTS

From the data collected in Table 3 it results that for all floors the weaker direction is the y-direction, where the resistant area of masonry piers is less. In relation to this direction, the ground floor appears to be more resistant than the other floors thanks to the higher thickness of walls. It also results that the prevalent failure mechanism of the piers is due to bending ($\xi_{yi} = 0.8$), with the exception of the top floor ($\xi_{y4} = 1$). 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 SIVARS system according to the piers slenderness and the loads applied on them.

A first difference between the old guidelines [14] and the new ones [15] concerns the evaluation of the shear strength at each floor and therefore the influence of the non dimensional parameter ζ_i and κ_i, introduced with the alignment to NTC 2008. In particular, at the top level in the weaker *y*-direction, it is evident that, being $\zeta_{y4} = 1$ and $\kappa_4 = 0.4$, the value of the shear strength representative of the base shear capacity of the building, is 2.5 times higher than the value obtained with the old guidelines. Besides, the influence of the parameter κ_i appears to be more relevant for the higher levels. In fact, at the first level in the *x*-direction the parameters are $\zeta_{x1} = 1$ and $\kappa_1 = 1$, so that the values of the shear strength related to the two versions of the guidelines are coincident.

Moreover, the ground accelerations corresponding to the two values of the base shear capacity have different results due to different ways to calculate the participant mass fraction e^*, which is 0.8384 and 0.25 for the new and the old guidelines, respectively. While this aspect should be deeper investigated, the two obtained safety indices in terms of acceleration belong to the same order of magnitude and declare that the building is rather unsafe with reference to the SLV limit state.

Another interesting difference between the two versions of the guidelines is evident in the procedure by which the ground acceleration corresponding to the capacity of the building is evaluated. The new guidelines, in fact, require the identification of the return period and all the other parameter of the seismic hazard related to this acceleration so that a different shape of the response spectrum is connected with the capacity of the building. Conversely, the capacity parameter given by the old guidelines in terms of ground acceleration is related to the shape of the reference response spectrum of the site.

Regarding the last remark, it is worth highlighting that the identification of a different return period of the seismic parameters connected with the capacity allows to obtain additional information useful in order to define a classification of the cultural heritage on the base of the exposition to seismic risk. In fact, with reference to the return period of the seismic action corresponding to the building capacity, it is possible define a more consistent value of the nominal lifetime, representing the validity period of the seismic safety assessment. The nominal lifetime corresponding to the building capacity can be obtained as $V_{N,SLV} = - T_{SLV} \times \ln(1-P_{VR})/C_u$, where P_{VR} for the limit state SLV is 0.1. For Pelella Palace the value of $V_{N,SLV}$ is about 7 years, consistent with the safety index of 0.42.

4 CONCLUSIONS

In this work the seismic vulnerability of a case study of historical masonry building located in Naples has been investigated by means of the first level of assessment (LV1) provided by the Italian Guidelines on Cultural Heritage.

The earlier version of these guidelines was implemented on line by the Ministry of Cultural Heritage and Activities (MiBAC) by means of the informative system named SIVARS, which is currently accessible through the web by institutional authorization. The result obtained using this system is expressed in terms of a sole seismic safety index summarizing the comparison between seismic demand and capacity.

Although the updated version of the Italian guidelines is based on the same approach for the assessment of the seismic vulnerability, it provides two safety indices, 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 define a more consistent value of the nominal lifetime, representing the validity period of the seismic safety assessment.

A perfect agreement between the two versions 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 indices in terms of acceleration belong to the same order of magnitude and denote that the masonry building under study is rather unsafe with reference to the achievement of the limit state of preservation of life (SLV). However, some differences were found about the calculation of the base shear capacity and the corresponding ground acceleration.

Lastly it is worth highlighting that the LV1 approach appears to be more conservative than other methods, e.g. nonlinear static analysis (pushover), 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. In addition, the safety parameter is more meaningful in terms of risk classification than in terms of structural response characterization.

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