Seismic analysis methods of ancient masonry churches in Matera

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ABSTRACT
Different seismic assessment methods for masonry buildings has been developed in Italy starting from the numerous damage surveys performed immediately after the recent earthquakes. These methods differ from each other for the number and completeness of available information as the structural details of primary elements and the material properties.
In this paper a critical review of the seismic assessment methods adopted by the Italian Guidelines on Cultural Heritage is presented. At this aim, some masonry churches located in the *Sassi di Matera* UNESCO site are analysed. The study compares the obtained results and discusses the utility of the applied methods, in relation to the possible results that can be achieved.

1 INTRODUCTION

The existing masonry churches are characterized typically by inefficient or absent connections among vertical walls, and among walls and horizontal elements (such as vaults and roofs). This determines, under lateral actions, the activation of some local overturning regarding single parts of the structure (also named macro-elements). This conclusion emerges from the observation of masonry ancient churches' damages occurred during the recent earthquakes (Friuli 1976, Irpinia 1980, Umbria and Marche 1997, Molise 2002, l'Aquila 2009, Umbria 2016), that have been collected and linked to the seismic action in terms of macro-seismic intensity and peak ground acceleration (PGA). In such a case, a global analytical prediction would be inappropriate since the collapse, rather to be widespread within the structure, is concentrated at the most vulnerable response mechanism. Therefore, in these cases the use of local models for the seismic analysis of existing masonry churches is more adequate since they provide more realistic results, especially in terms of activation threshold of mechanism.

The Italian Guidelines for the Reduction of Seismic Risk on Cultural Heritage (DCCM 2011) proposed three different levels of evaluation, called LV1, LV2 and LV3, having an increasing level of complexity, precision and reliability of the obtainable results. As the aim of this work is to assess the vulnerability of the churches sited in the *Sassi di Matera* UNESCO site, the analysis is focused on the church typology.

The LV1 method is based on a qualitative approach for evaluating the seismic safety at a territorial scale and is, therefore, mainly aimed at establishing a priority list for interventions plans on Cultural Heritage. It analyses the vulnerabilities of all possible church macro-elements, providing a global vulnerability index linked, through empirical relationships, to the ground acceleration corresponding to the achievement of the considered ultimate limit state. The LV2 method permits, by collecting a higher level of information with respect to the LV1 method, of defining the degree of vulnerability of single portions of the structure (macro-element) by determining the ground acceleration related to the supposed ultimate condition (commonly the failure of the macro-element). Finally, the LV3 method, represents the most complex method of analysis since it refers to global model of the churches, allowing of describing properly the distribution of vertical loads and the interaction among the local response mechanism until the first failure.

In this paper the LV1 and LV2 methodologies provided by the Italian Guidelines (DCCM 2011) are applied to some churches chosen as case studies and located into a moderate seismic-prone area. In particular, the six churches investigated, falling within *The Sassi and the Park of the Rupestrian Churches of Matera*, recognized as Cultural World Heritage by UNESCO since 1993, are: San Giovanni Battista, San Pietro Caveoso, San Rocco, San Francesco d'Assisi, Sant'Agostino and Santa Maria de Armenis. The case studies are firstly examined one by one by applying the LV1 and LV2 methodologies proposed by the Italian Guidelines. Then the results are discussed and compared among them.

2 SEISMIC RISK ASSESSMENT BY THE ITALIAN GUIDELINES

2.1 LV1 method (DCCM 2011)

The LV1 method defines a vulnerability index for the churches typology, based on the analysis of 28 collapse mechanisms of different structure portions or macro-elements (Figure 1).

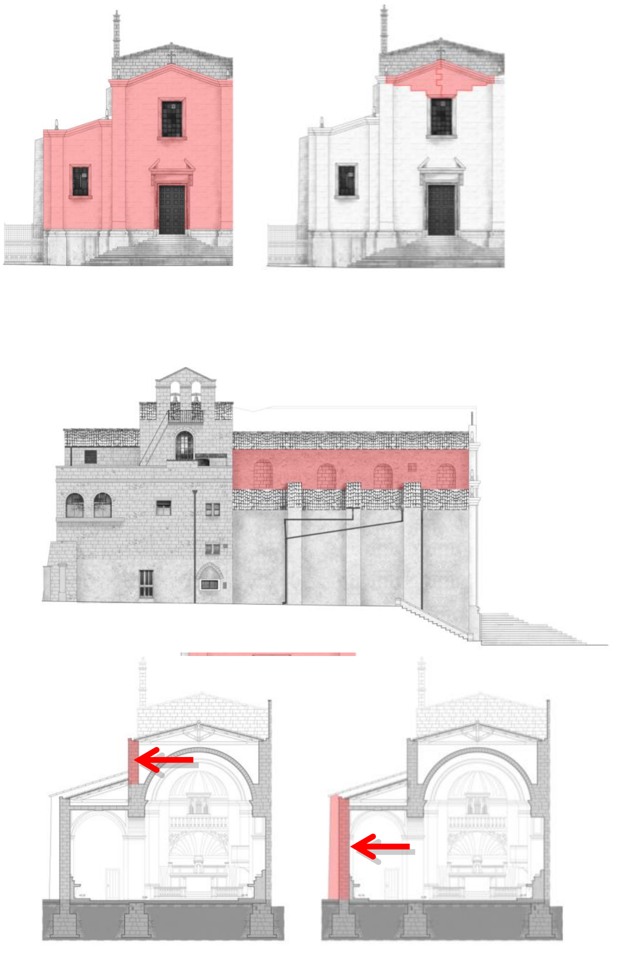

Figure 1. Collapse mechanisms in the church of San Rocco: façade and pediment overturning; mechanisms related to the roof elements in the lateral wall; and lateral wall overturning (Chietera 2017).

These collapse mechanisms have been proposed after analysing the observed damage of about 3000 churches linked to different seismic intensities, and by considering a vulnerability index taking into account the fragilities and seismic-resistant devices of each macro-element (Lagomarsino e Podestà 2005). The collapse mechanisms taken into account are: regarding the façade: overturning and in-plane mechanisms, mechanisms in the top of the façade and in the narthex; in the nave: transversal and longitudinal response, columns longitudinal response, and the vault response in the central and lateral naves; regarding the transept, chapels, presbytery and the apse: overturning and shear mechanisms in walls, and vaults response; related to the roof components: mechanisms in the lateral walls of the nave, in the transept, apse or presbytery. Other mechanisms analysed are related to: interactions in proximity of irregularities in plan or elevation; projections (gable, pinnacles, statues, etc.); the presence of a bell tower, dome – drum or lantern.

The vulnerability index is expressed by Eq. (1), where v_{ki} is the score of the fragility indicators and v_{kp} is the score of the seismic-resistant devices. Each of them has a score ranging from 0 to 3 regarding the degree of vulnerability or efficiency, respectively, and ρ_k is the weight of each collapse mechanism:

$$i_v = \frac{1}{6}\frac{\sum_{k=1}^{28}\rho_k\left(v_{ki}-v_{kp}\right)}{\sum_{k=1}^{28}\rho_k}+\frac{1}{2} \tag{1}$$

After, the Eq. (2) and Eq. (3) allow to calculate the values of ground acceleration corresponding to the damage limit state (SLD) and the life-safety limit state (SLV).

$$a_{SLD}S = 0.025\cdot 1.8^{2.75-3.44 i_v} \tag{2}$$

$$a_{SLV}S = 0.025\cdot 1.8^{5.1-3.44 i_v} \tag{3}$$

The relation between the vulnerability index and the ground acceleration corresponding to the life-safety and damage limit states is shown in Figure 2.

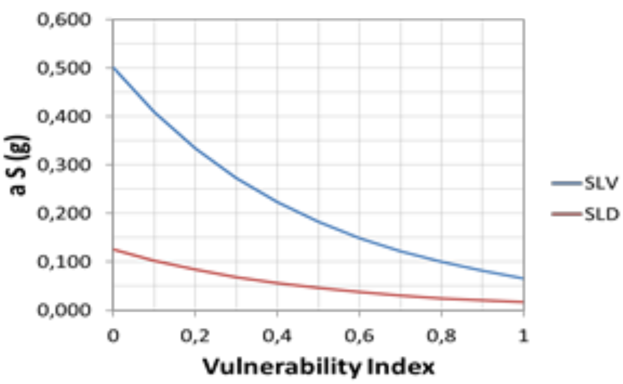

Figure 2. Graphic of vulnerability index and ground acceleration regarding the life-safety and damage limit states.

By knowing the ground acceleration (a_{SLV} or a_{SLD}), the seismic safety index (IS) and the acceleration factor (F_a) can be calculated. The seismic safety index is estimated by the relation between the return period (T_{SL}) of the seismic action which provokes the generic limit state, and the corresponding return period of reference ($T_{R,SL}$), related to the earthquake expected on the site (Eq. 4). The acceleration factor (F_a) is calculated by the relation between the acceleration which provokes the generic limit state (a_{SL}) and the acceleration expected on the site ($a_{g,SL}$) (Eq. 5). Therefore, the building is in a safe condition when the IS and F_a are greater than or equal to 1.

$$IS = \frac{T_{SL}}{T_{R,SL}} \tag{4}$$

$$F_a = \frac{a_{SL}}{a_{g,SL}} \tag{5}$$

The return period ($T_{R,SL}$) of the obtained ground acceleration ($a_{g,SL}$) can be calculated as follows (Eq. 6).

$$T_{R,SL} = T_{R1} \cdot 10^{\alpha} \tag{6}$$

where α is calculated by the (Eq. 7):

$$\alpha = \left[\log\left(a_g\right) - \log\left(a_{g,1}\right)\right] \cdot \frac{\log\left(\frac{T_{R2}}{T_{R1}}\right)}{\log\left(\frac{a_{g,2}}{a_{g,1}}\right)} \tag{7}$$

where the subscript 1 refers to the data of return period and acceleration immediately lower than $a_{g,SL}$, and with the subscript 2, those immediately above, in reference to the table provided by the Annex B of the NTC-2008. Thus, the nominal life, which in accordance with the Italian Guidelines (DCCM 2011) corresponds to the number of years in which the church can be securely used, provided with ordinary maintenance, is given by (Eq.8):

$$V_N = -\frac{T_{R,SL}}{C_U} \ln\left(1 - P_{VR}\right) \tag{8}$$

In the previous Eq. (8), in the case of the life-safety limit state, the probability of exceeding of the structure (P_{VR}) is equal to 10% and C_U represents the importance factor, in accordance with the NTC-2008 (D.M. 2008).

2.2 LV2 method (DCCM 2011)

The LV2 assessment level regards single parts of the structure, and it is suitable for investigating the efficiency of some interventions, without significantly altering the structural behaviour of the whole system. As historic masonry buildings usually present connection deficiencies between walls, floors and roofs elements, the most vulnerable failure mechanisms consist of out-of-plane overturning of rigid bodies, caused by the loss of stability of structurally independent parts. Therefore, the kinematic analysis is used to perform the macro-elements vulnerability assessment by the means of rigid bodies modelling. The identification of macro-elements depends on the knowledge of the quality of the connections between the parts, not only considering constructive joints but also the presence of crack patterns caused by decay or previously occurred seismic events. For the same purpose, the survey of the changes or alterations of all over the structure during the past is useful to individuate autonomous structural parts.

The kinematic approach is addressed by the application of the theorem of equilibrium limit analysis (Heyman 1997), based on a preliminary identification of the collapse mechanism, which transforms the structure in a kinematic mechanism by the introduction of a sufficient number of hinges or sliding planes. Each block is considered under vertical loads and horizontal seismic forces, which are proportional to the vertical loads by a coefficient λ. In according with the hypothesis of masonry non-resistant to traction, with infinite resistance to compression and rigid blocks, the load collapse multiplier λ_0 that causes the loss of the equilibrium is calculated through the application of the principle of virtual work (Lagomarsino e Podestà, 2005:17).

Once the proper quality of the masonry has been ensured, for excluding the possibility of the wall crumbling, at every possible local collapse mechanism, a state of virtual displacements with the relative multiplier λ (Eq. 9) are settled, by applying the theorem of virtual works, which represents the load amplification caused by the earthquake, which determines the activation of the hypothesized kinematism (Vinci 2014a:4):

$$\lambda\left(\sum_{i=1}^{n} P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j}\right) - \sum_{i=1}^{n} P_i \delta_{y,i} - \sum_{h=1}^{o} F_h \delta_h = L_{fi} = 0 \tag{9}$$

where:

- n is the number of all the load forces applied to the different blocks of the kinematic chain;
- m is the number of load forces not directly related to the blocks, which masses, due to the seismic action, generate horizontal

forces on the elements of the kinematic chain, as they are not effectively transmitted to other parts of the building;
- o is the number of external forces, not associated with masses, applied to the different blocks;
- P_i is the generic weight force applied to the block;

of which:
- P_j is the generic weight force, not directly applied to the blocks, which mass, as a result of the seismic action, generates a horizontal force on the elements of the kinematic chain, as it is not effectively transmitted to other parts of the building;
- $\delta_{x,i}$ is the horizontal virtual displacement of the point of application of the i-th P_i load, assumed as positive if it acts in the same direction of the seismic action that activates the mechanism;
- $\delta_{x,j}$ is the horizontal virtual displacement of the point of application of the j-th P_j load, assumed as positive if it acts in the same direction of the seismic action that activates the mechanism;
- $\delta_{y,i}$ is the virtual vertical displacement of the point of application of the i-th P_i load, assumed as positive if upwards;
- F_h is the absolute value of the generic external force applied to a block;
- δ_h is the virtual displacement of the point of application of the h-th F_h external force, in the same direction of the external force and with a positive sign if the sense is discordant;
- L_{fi} is the work of potential internal forces.

The capacity of the structure is, thus, evaluated with respect to the hypothesized mechanism expressed in terms of the acceleration which activates it, calculated through a conversion process (10) applied to the load multiplier λ:

$$\alpha_0^* = \frac{\lambda \sum_{i=1}^{n+m} P_i}{M^* FC} = \frac{\lambda g}{e^* FC} \tag{10}$$

where:
- λ is the collapse multiplier;
- $\sum_{i=1}^{n+m} P_i$ is the sum of all the load forces applied which masses, due to seismic action, generate horizontal forces on the elements of the kinematic chain;
- FC is the confidence factor. If an infinite compression strength of the masonry is considered in the evaluation of the collapse multiplier λ (with a hinge around which the macro element rotates, on the outer edge), the confidence factor is assumed to be 1.35, a value corresponding to a low level of knowledge.
- e^* is the fraction of participating mass (Eq. 11):

$$e^* = \frac{gM^*}{\sum_{i=1}^{n+m}} \tag{11}$$

- M^* is the mass participating in the collapse mechanism, determined by Eq. (12):

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{x,i}^2} \tag{12}$$

where:
- g is the gravity acceleration;
- $\sum_{i=1}^{n+m} P_i \delta_{x,i}$ is the sum of all weight forces whose masses, due to seismic action, generate horizontal forces on the elements of the kinematic chain, multiplied by their respective horizontal displacements.

In the case studies, e* and M* are considered equal to 1 in favour of security.

The comparison between the acceleration which leads to the kinematism, before and after the retrofitting intervention, allows to assess its effectiveness; whereas a ground acceleration expected at the site lower than the necessary for the kinematic mechanism activation, means that the seismic improvement is unnecessary.

The expected seismic action, on the other hand, is estimated through a probabilistic method and it is expressed in terms of maximum horizontal ground acceleration in a given period of time, by using the software *Spectra Response*, developed by the Higher Council of Public Works (2008), which provides seismic hazard parameters relative to any geographical location. Once the maximum ground acceleration is given by the software, it is possible to determine the seismic demand for which the whole building or each mechanism must be verified through the following equations, distinguishing the demand when the hinge is formed in (Eq. 13) or over (Eq. 14) the ground level.

$$\alpha_{0,rif}^* \geq \frac{a_g(P_{VR})S}{q} \tag{13}$$

$$\alpha_{0,rif}^* \geq \frac{S_e(T_1)\psi(Z)\gamma}{q} \tag{14}$$

In the Eq. (13), the ground acceleration value could be amplified by a coefficient dependent on the characteristics of the subsoil ($S=S_S \cdot S_T$) and it is proportional to the structure factor (q), which will assume an unitary value for precautionary reasons. The Eq. (14), instead, considers the hinges formation over the ground level, therefore, the fundamental period of vibration (T_1) has to be calculated through the Eq. (15) recommended by Lagomarsino (2005) for the churches typology, where H is the height of the structure up to the eaves line:

$$T_1 = 0{,}07H^{\frac{3}{4}} \tag{15}$$

The author proposed a value for the modal participation coefficient (γ) equal to 1.1 as well (Lagomarsino e Podestà 2005). After, the value of T_1 is used to find the equation for calculating $S_e(T_1)$ through the software *Spectra Response.* The first mode of vibration (ψ (Z)) is obtained by dividing the high of the hinge formation (Z) with respect to the foundation, by the total building height (H) (Eq. 16):

$$\psi(Z) = \frac{Z}{H} \tag{16}$$

As regards the LV3 evaluation level, it assesses the global seismic response of the building, assuming a better condition of the connections between the structural elements, through the use of a detailed global modelling, which considers the interaction between the structural elements. As this analysis assumes the box-behaviour of the building, the resulting damage is demonstrated on a diffuse way, which greater dissipation of energy shows a greater capacity of the structure, especially for facing seismic events of considerable entity.

As an alternative to a detailed global modelling, it is possible to discretize the structure in significant macro-elements and then systematically verify the stability of each of them under the horizontal actions by the local model LV2, considering the possibility of collapse reached by the loss of the stability of the parts.

3 APPLICATION TO SIX CHURCHES IN THE SASSI DI MATERA UNESCO SITE

In order to apply both procedures, six churches sited in *Sassi di Matera* were analysed (Figure 3). Five of them are built with calcarenite (limestone) masonry: the church of Sant'Agostino, San Rocco and San Francesco d'Assisi have a one-nave basilica floor plan configuration, and the churches of San Pietro Caveoso and San Giovanni Battista have a three-naves configuration. Regarding the roof structure, Sant'Agostino, San Rocco and San Giovanni Battista have limestone vault systems, while San Francesco d'Assisi and San Pietro Caveoso have wooden structures and limestone vaults roofing. The sixth church is Santa Maria de Armenis, that differently from the others is excavated in the rock with a three-leaf masonry façade scarcely connected to the rock cave.

3.1 LV1 method application

As the case studies correspond to churches and they have a frequent and sometimes crowded use, the C_U has been assumed equal to 1.5 corresponding to the class III (DM 2008).

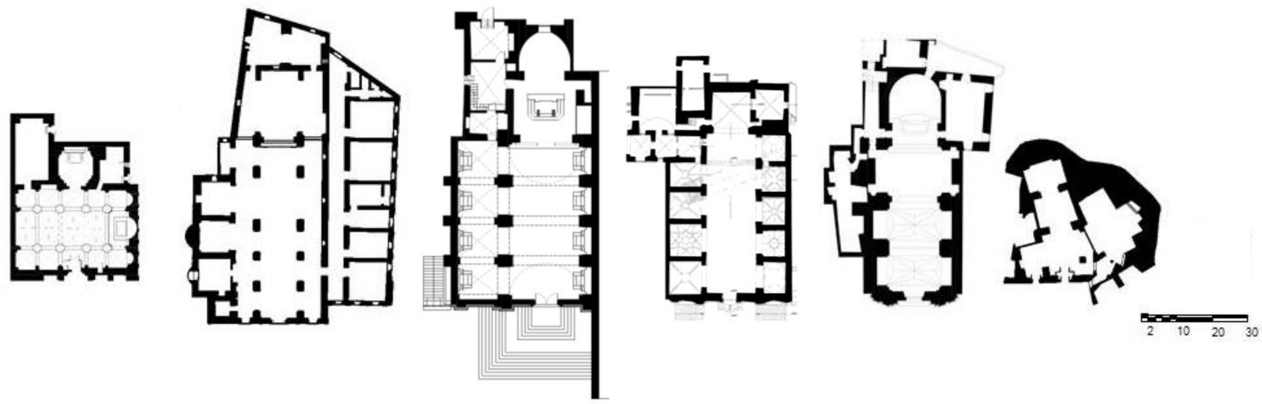

Figure 3. Churches floor plan (from left to right): San Giovanni Battista, San Pietro Caveoso, San Rocco, San Francesco d'Assisi, Sant'Agostino and Santa Maria de Armenis. Reference: Superintendence for the Architectural and Landscape Assets in Italy, 2016; La Scaletta 1960; and Chietera, 2017.

Regarding the nominal life (V_N) two reference periods have been considered that are: 50 years, which is the reference time for the new buildings; and 20 years, which is the minimum required by the Italian Guidelines (DCCM 2011).

Therefore, for defining the seismic action for the life-safety limit state two reference periods

(V_R) of 35 and 75 years (V_R=V_N·C_U) have been considered. Moreover, in this study the rock subsoil has been assumed of category A, and the coefficient for the amplification due to topographical condition S_T has been considered equal to 1.2, only in the churches of Sant'Agostino and San Pietro Caveoso, since they are sited on a ridge, while for all the other cases S_T is equal to 1.0 (flat surface).

The obtained results are summarized in Table 1, where the return period ($T_{R,SLV}$) for the expected seismic action (life-safety limit state) results is equal to 285 and 712 years.

Table 1. Scores of the parameters to define the numerical seismic response assessed by the LV1 and LV2 methods (DCCM 2011).

	Parameters	San Giovanni	S. Pietro Caveoso	San Rocco	San Francesco	Sant' Agostino	S. Maria Armenis
LV1	i_v	0,44	0,49	0,57	0,65	0.71	1
	$a_{SLV}S$	0,210 g	0,186 g	0,158	0,134 g	0,119 g	0,066 g
	$a_{g\,SLV}$	0,121 g	0,136 g	0,121 g	0,097 g	0,168 g	0,123 g
	T_{SLV}	1538	1028,3	635,7	405,1	301	5,82
	V_N (years)	108,03	72,23	44,65	28,45	21,14	5,82
	IS (T_{SLV} / $T_{R,SLV}$) (V_N=50; $T_{R,SLV}$=712)	2,16	1,44	*0,89*	*0,57*	*0,42*	*0,01*
	IS (T_{SLV} / $T_{R,SLV}$) (V_N=20; $T_{R,SLV}$=285)	5,40	3,61	2,23	1,42	1,06	*0,02*
	Fa (a_{SLV} S / $a_{g\,SLV}$)	1,74	1,37	1,31	1,38	*0.71*	*0.68*
LV2	Mechanisms applied on ground level ($a_{g\,SLV}$)	$a_0^* = \frac{\lambda \cdot g}{e \cdot F_c} \geq \frac{a_g(P_{VR})S}{q}$					
	Façade overturning	0,162	*0,065*	*0,050*	*0,059*	*0,076*	*0,085*
	Façade right-side overturning	*0,074*	-	-	-	-	-
	Façade left-side overturning	0,149	-	-	-	-	*0,077*
	Entrance arcade overturning	*0,051*	-	-	-	-	-
	Façade compound overturning	-	-	*0,057*	0,115	-	*0,087*
	Façade compound overturning 30°	-	-	-	*0,068*	-	-
	Lateral façade/apse overturning	*0,107*	-	-	*0,080*	-	-
	Partial lateral facade overturning	*0,033*	-	-	-	-	-
	Bottom of the lateral facade overturning	-	-	0,161	-	-	-
	Longitudinal response of the colonnade-rigid band	0,167	-	0,156	-	-	-
	Longitudinal response of the colonnade-weak band	-	0,271	*0,106*	0,322	-	-
	Transversal vibration of the nave (right)	0,185	*0,071*	*0,108*	*0,046*	*0,161*	-
	Transversal vibration of the nave (left)	-	-	-	*0,053*	-	-
	Transversal vibration of the triumphal arch	-	0,166	-	0,650	-	-
	Bell tower overturning	-	0,275	-	-	-	-
	Façade shear mechanism – global diagonal crack (1)	-	0,611	-	0,871	0,322	-
	Façade shear mechanism – global diagonal crack (2)	-	0,480	0,557	-	-	-
	Façade shear mechanism– base diagonal cracks with central vertical crack	-	-	0,353	0,408	0,266	-
	Mechanisms applied over the ground level (a g SLV)	$a_0^* = \frac{\lambda \cdot g}{e \cdot F_c} \geq \frac{S_e(T_1)\psi(Z)\gamma}{q}$					
	Top of the facade overturning	*0,064*	0,173	-	0,103	*0,090*	-
	Seismic demand Top of the facade overturning	0,153	0,164	-	0,059	0,101	-
	Top of the lateral facade overturning	-	-	0,390	-	-	-
	Seismic demand Top of the lateral facade overturning	-	-	0,114	-	-	-
	Breakout of the pediment	-	0,450	0,252	-	0,350	-
	Seismic demand Breakout of the pediment	-	0,214	0,150	-	0,138	-
	Pediment overturning	0,173	0,788	-	-	0,477	-
	Seismic demand: Pediment overturning	0,097	0,244	-	-	0,156	-

As it is easy to note, in the case of V_N=20 years only the church of Santa Maria de Armenis has a seismic safety index (IS) significantly less than one. For this church the same situation is found when V_N=50 years, together with the church of Sant'Agostino. It must be remarked that since the Santa Maria de Armenis church is an excavated church, only the façade overturning collapse mechanism has been considered in this study.

By comparing the obtained results (Table 1) it can be noted that, in the analysed cases the most vulnerable macro-elements are: the damage at the top of the façade due to the presence of gables, the vaults mechanisms of the lateral aisles due to their irregular shapes in conjunction with the lack of steel chains, and the apse overturning due to the vaults thrusts with the lack of contrast elements as buttresses (Laterza et al. 2017a, Laterza et al. 2017b).

3.2 *LV2 method application*

The LV2 method proposed by the Italian Guidelines (DCCM 2011) has been applied to the case studies in order to evaluate the seismic response, in terms of horizontal acceleration, activating the macro-element considered. The obtained results are summarized also in the form of histograms (Figure 4a, 4b), where also are reported the results obtained with the LV1 method in terms of acceleration and seismic demand ($a_{g,SLV}$).

The results obtained are reported in Table 1, where the q-factor has been assumed equal to one. It must be noted that, in according to this method, the lowest accelerations activating a macro-element have been obtained in the case of façade overturning and in the partial façade top overturning, confirming that the façade is one of the most vulnerable macro-elements.

In the Figure 4a and 4b are reported only the accelerations for activating the life-safety limit state referring to macro-elements having their hinge at ground level (shown in pink), and the ones with their hinge above the ground level (shown in red).

Regarding the collapse mechanisms which hinge are formed above the ground level, the accelerations for activating the pediment collapse mechanisms, are higher than the accelerations activating collapse mechanisms at ground level. This is mainly due to the pediment´s geometry, as they have a low height, a low slenderness and a minor distance for calculating the overturning motion as well. Hence, the pediments are not affected by the thrust of vaults and roof elements. Instead, the mechanisms at the top of the façade are most of them vulnerable, which responds to the presence of gables, or mechanisms in the façade top, when there are irregularities in the geometry or thickness in the same façade.

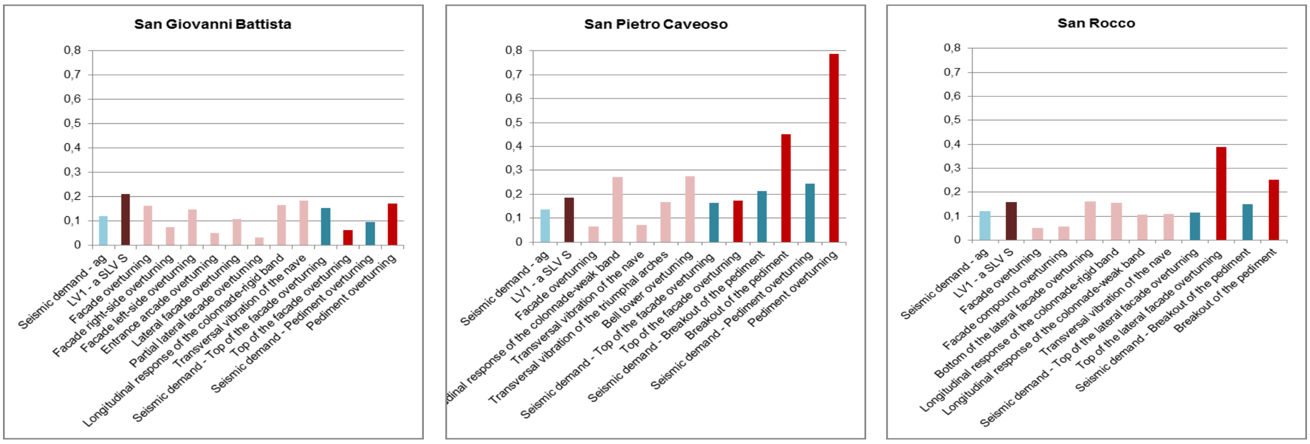

Figure 4a. LV1 and LV2 histograms regarding the current capacity of the churches of San Giovanni Battista, San Pietro Caveoso and San Rocco, expressed in terms of the acceleration of the out-of-plane mechanism activation and the seismic demand.

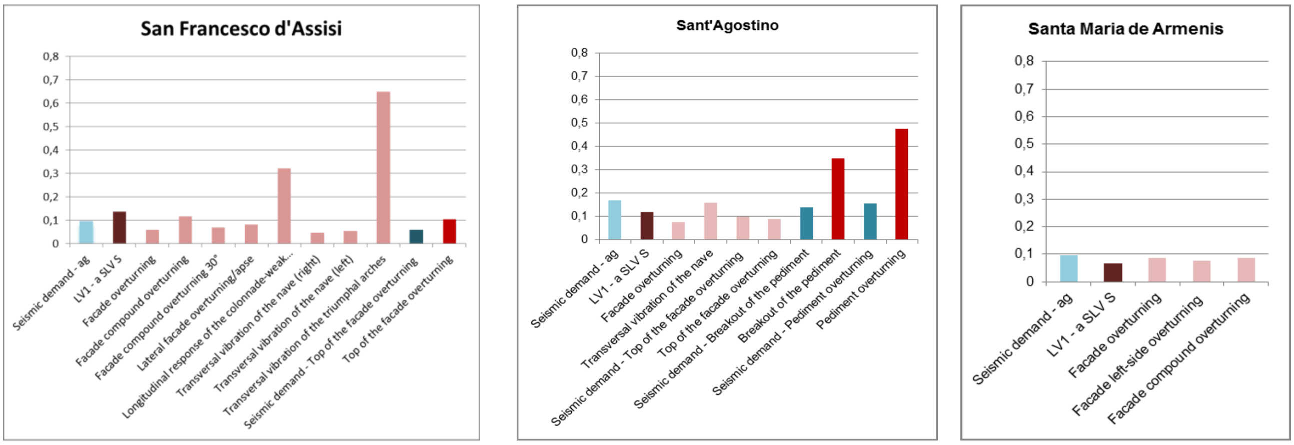

Figure 4b. LV1 and LV2 histograms regarding the current capacity of the churches of San Francesco d'Assisi, Sant'Agostino and Santa Maria de Armenis, expressed in terms of the acceleration of the out-of-plane mechanism activation and the seismic demand.

4 CONCLUSIONS

The Italian Guidelines for the assessment and mitigation of the seismic risk of the cultural heritage (DCCM 2011) define an acceptance criteria by tolerating a lower security level for the cultural heritage then the new building, through the definition of a reduced nominal life, even less than 50 years. This acceptance criteria protects the construction in probabilistic terms for a lesser number of years, but enables less invasive interventions, and allows interventions planning based on the nominal life.

Regarding the obtained results, the most recurrent fragile macro-elements found with the LV1 method qualitative analysis are: the damage at the top of the façade due to the presence of gables; the shear mechanisms in the façade due to the high slenderness; the vaults mechanisms of the lateral aisles due to their irregular shapes and lack of steel tensors; and the apse overturning due to the vaults thrust and the lack of contrast elements as buttresses. With the LV2 method, the fragility of the façade macro-element has been confirmed, as for all the case studies, the calculated accelerations activating the mechanisms regarding the façade or the top of the façade are vulnerable, even in a moderate seismic-prone area such as Matera. The vulnerability of the lateral aisles has been also confirmed by the results of the transversal vibration of the nave. In particular, regarding the less vulnerable church, San Giovanni, the LV2 method highlights its vulnerability to the apse overturning mechanism, as the church's access has been changed and now the lateral façade corresponds to the previous apse.

The application of the kinematic limit analysis is suitable for evaluating the churches resistant capacity, as the modelling for a structural finite element analysis presents considerable difficulties requiring extensive computational costs. Thus, this analysis approach with macro-elements, taking into account the thrusts caused by vaults and roofs, may be considered more representative than global models.

Nevertheless both procedures are not comparable regarding the calculated acceleration which activates the life-safety limit state, because the LV1 method provides an average acceleration of the whole church based on qualitative observations, while the LV2 provides an specific acceleration which activates the life-safety limit state for each macro-element, being the second method more reliable.

Finally, both methods are complementary regarding the characterization of the most fragile macro-elements. The LV1 method allow to define a global numerical priority and to identify the most fragile macro-elements as a result of the observational qualitative method, whereas the LV2 method performs an evaluation to define the specific numerical response to each collapse mechanism, which allow to propose retrofitting projects, both without performing invasive tests in the historic masonry buildings.

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