European Community on
Computational Methods in
Applied Sciences

ECCOMAS
Thematic Conference

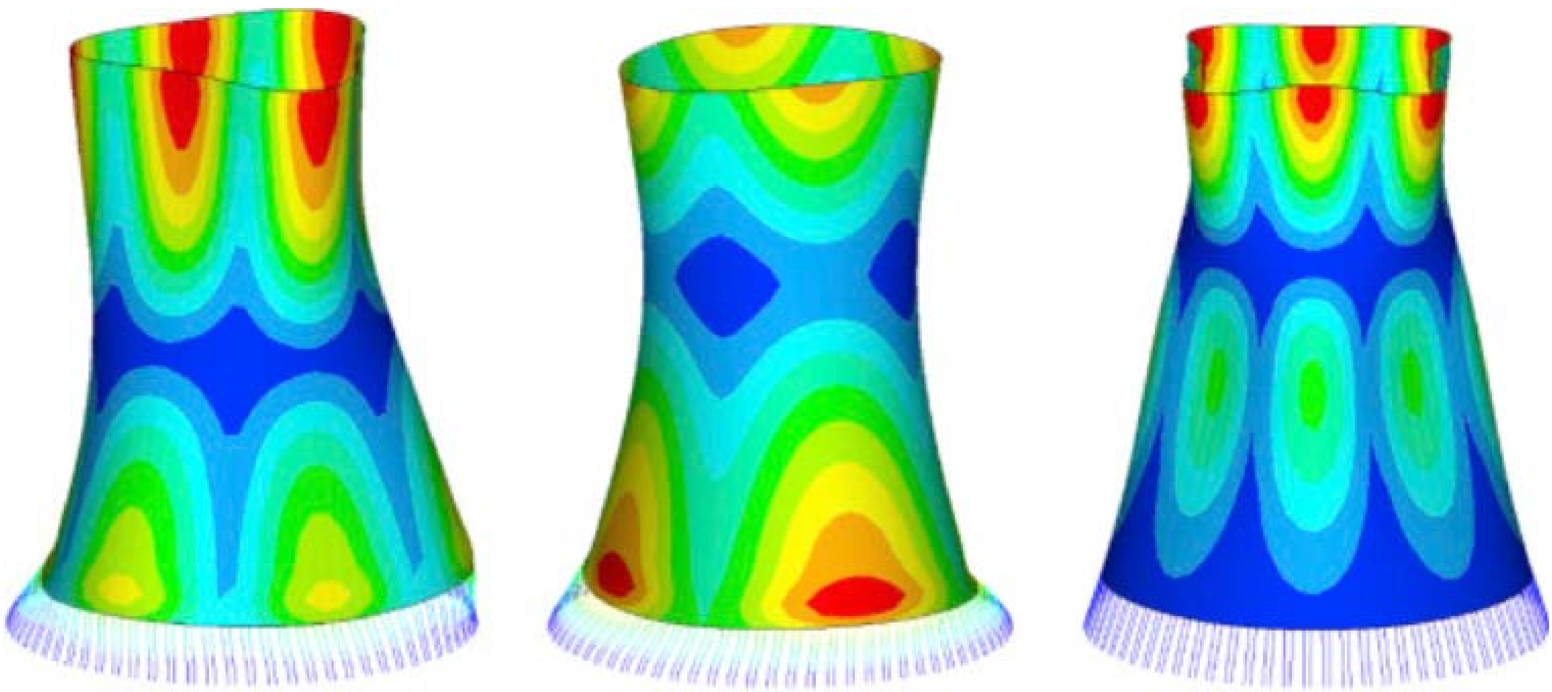
COMPDYN 2019

7th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering

PROCEEDINGS

Volume II

M. Papadrakakis, M. Fragiadakis (Eds)



An IACM Special Interest Conference

COMPDYN 2019
Computational Methods in Structural Dynamics and Earthquake Engineering

Proceedings of the 7th International Conference on Computational
Methods in Structural Dynamics and Earthquake Engineering
Held in Crete, Greece
24-26 June 2019

Edited by:

M. Papadrakakis
National Technical University of Athens, Greece

M. Fragiadakis
National Technical University of Athens, Greece

A publication of:

Institute of Structural Analysis and Antiseismic Research
School of Civil Engineering
National Technical University of Athens (NTUA)
Greece

COMPDYN 2019

Computational Methods in Structural Dynamics and Earthquake Engineering

M. Papadrakakis, M. Fragiadakis (Eds)

First Edition, October 2019

ISBN (set): **978-618-82844-5-6**
ISBN (vol II): **978-618-82844-7-0**

COMPDYN 2019
7th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Crete, Greece, 24–26 June 2019

VERTICAL COMPONENT OF THE SEISMIC ACTION: AMPLIFIED VULNERABILITY OF EXISTING MASONRY BUILDINGS

Massimo Mariani[1], **Francesco Pugi**[2] **and Alessio Francioso**[3]

[1] Studio Ricerche Applicate
06123 Perugia, Italy
ricercheapplicate@libero.it

[2] Aedes Software: Ricerca e Sviluppo
56028 San Miniato Basso, Pisa, Italy
info@aedes.it

[2] National Technical University of Athens
15780 Zografou, Athens, Greece
alessiofrancioso@gmail.com

Abstract

The recurrent earthquakes which recently stroked the center of Italy caused severe damages to the built heritage showing extensive disaggregation mainly due to poor masonry quality. In this phenomenon, the vertical seismic accelerations played a crucial role. The aim of this study is to define a methodology for considering the effects of the vertical seismic component in the seismic assessment of masonry buildings through kinematic and pushover analysis. In pushover analysis, inertial forces caused by the vertical accelerations are considered in the safety verifications of the elements resulting in a drop of the global resistance and displacement capacity of the structure. The proposed methodology is applied to the case study of a two-stories masonry building for several levels of seismic action and for different levels of masonry quality. The results of the analyses performed with and without the vertical seismic component are compared and discussed. Then, the methodology is applied to the seismic assessment of a real masonry building through pushover analysis of the current state and evaluation of possible strengthening measures. The effect of the vertical seismic component in both configuration of the building is discussed.

Keywords: Seismic assessment, Existing Masonry Buildings, Vertical Component, Pushover Analysis, Kinematic Analysis, Equivalent Frame Method

1 INTRODUCTION

The severe damages caused by recent earthquakes in the center of Italy confirmed the high vulnerability of existing masonry buildings (Figure 1). The damage investigation carried out in recent studies highlighted, among other aspects, the crucial role played by the vertical seismic component in the loss of ductility and consequent masonry disaggregation [1].

Figure 1. Extensive damages caused by earthquakes at Pescara del Tronto (top), Accumoli (bottom left) and Castelluccio di Norcia (bottom right)

Figures 2-4 show the accelerograms related to the earthquakes of L'Aquila 2009, Emilia 2012 and Norcia 2016. The graphs represent the horizontal (red) and vertical (blue) component of the seismic action in the first 5 second of the seismic excitation. It is evident that in all these earthquakes the vertical seismic component played an important role.

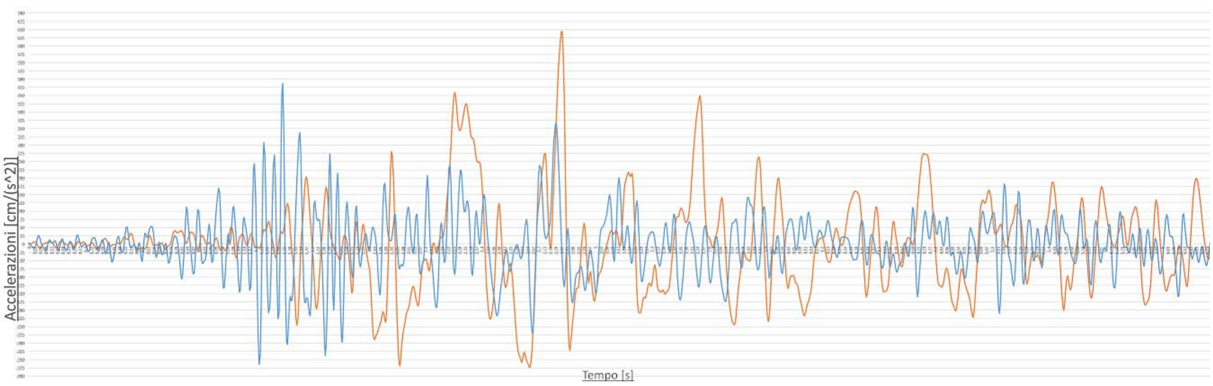


Figure 2. L'Aquila 2009. Horizontal and vertical component of seismic action

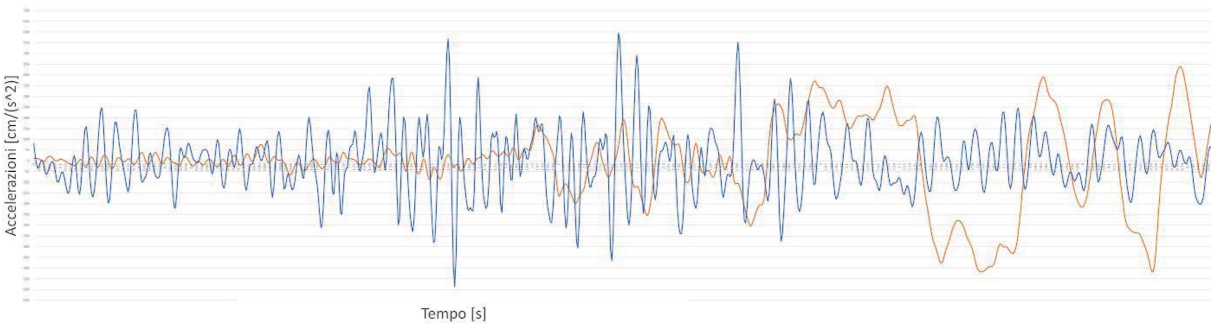


Figure 3. Emilia 2012. Horizontal and vertical component of seismic action

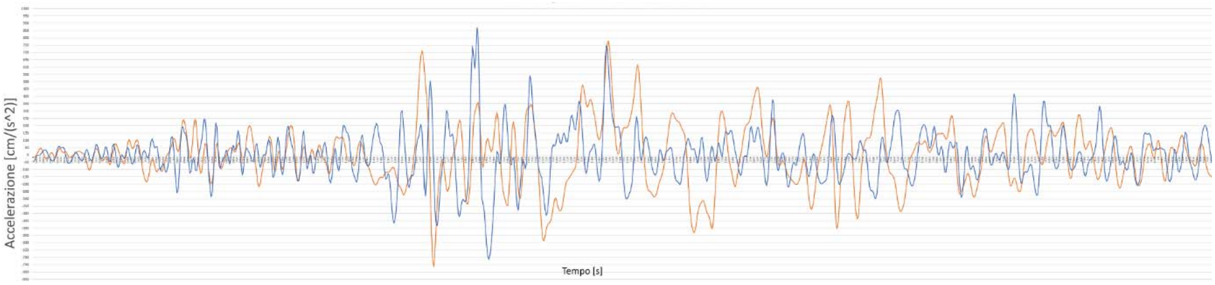


Figure 4. Norcia 2016. Horizontal and vertical component of seismic action

Among the damages caused by the earthquakes, several masonry buildings showed pure shear failures with clear separation of horizontal mortar beds and sliding for several centimeters (Figure 5). These types of failures typically occur where the walls feature variation of horizontal cross section due to the presence of openings or floors, or for discontinuity in elevation. The phenomenon can be explained considering the combined action of vertical acceleration that lightened the structure and horizontal acceleration that caused shear action in masonry piers.

Figure 5. Pure shear failures (courtesy of Alessandro De Maria)

Therefore, the effects of the vertical seismic component on the seismic capacity of existing masonry building should be taken into account. Previous studies already confirmed that in other structural typologies failure may ensue due to direct tension or compression as well as due to the effect of vertical motion on shear and flexural response [2].

The objective of this work is to define a methodology viable for professionals for considering the vertical seismic component in the seismic assessment of masonry building through kinematic and pushover analysis. In Pushover analysis the effects of the vertical seismic component are modelled through a field of vertical inertial forces resulting from modal response spectrum analysis. During the incremental analysis, safety verifications applied to the elements are carried out combining the internal actions induced by the vertical forces with those arising from static and incremental horizontal loading.

The proposed methodology is applied to the case study of a two-story masonry building considering different levels of masonry quality and several levels of seismic intensity.

Then, further aspects of the methodology are introduced in order to make it viable for professional application in the assessment of real masonry buildings: (a) effects of compression and decompression cycles due to vertical seismic excitation, (b) capacity of the structure in terms of PGA considering the variability of the vertical seismic effects. This defines an algorithm for considering the vertical seismic effects in any professional software able to perform kinematic and pushover analyses of masonry buildings.

The complete methodology is applied to the seismic assessment of a real masonry building evaluating the effects of possible strengthening interventions.

2 EFFECTS OF VERTICAL SEISMIC COMPONENT ON SEISMIC ASSESSMENT OF MASONRY BUILDINGS

The vulnerability of an existing masonry building is first and foremost conditioned by the quality of masonry itself. Under seismic loading, masonry of poor quality is very likely to develop phenomena of disaggregation which can hardly be assessed by any mechanical model. Therefore, the seismic assessment of masonry buildings cannot prescind from an accurate analysis of the masonry quality and only if disaggregation phenomena are prevented it makes sense to investigate other failure mechanisms.

The next source of vulnerability for masonry structures is associated to local mechanisms mainly due out-of-plane behavior of walls. The seismic response of the building is governed by such mechanisms when connections between orthogonal walls and between walls and floors are particularly poor. Only if connections are improved by proper devices (e.g. tie-rods), local mechanisms can be prevented, and a global behavior governed by the wall in-plane behavior can develop.

In summary, safety assessment of a masonry building could be performed in three consequential phases:

1. Analysis of masonry quality in order to prevent eventual phenomena of disaggregation and define adequate mechanical parameters
2. Analysis of local collapse mechanisms through kinematic analysis
3. Analysis of in-plane response of masonry elements through pushover analysis. The analysis may be applied to single walls or global structure depending on the quality of connections between the elements.

Seismic assessment of an existing masonry building highlights its vulnerability and allows to define an adequate plan of strengthening interventions. Therefore, in order to assess the real capacity of the structure it is important to consider all the effects of the seismic action.

The vertical component of the seismic action affects negatively all the phases of the seismic assessment: the vertical dynamic actions accentuate eventual masonry disaggregation and tend to worsen the capacity of the structural elements with respect to local and global failure mechanisms.

In the next paragraphs a methodology for considering the effects of the vertical seismic component in kinematic and pushover analysis is presented and applied to the case study of a two-story masonry building.

3 KINEMATIC ANALYSIS

According to current Standards, Linear Kinematic Analysis is performed with the following steps:

1. Definition of the collapse mechanism: axes of rotations, participating bodies, forces.
2. Calculation of the collapse multiplier of the seismic action and the correspondent spectral acceleration that activates the mechanism.
3. Calculation of the capacity in terms of PGA, that is the ground acceleration correspondent to the spectral acceleration activating the mechanism

In this context, the effects of the vertical seismic component may be modelled as a field of inertial forces directed upwards which tend to anticipate the activation of the mechanism. As a result, the collapse multiplier and the capacity in terms of PGA decrease.

Consider the mechanism in Figure 6, simple overturning of a masonry wall. The forces involved in the mechanism are the self-weight of the rigid body P, the horizontal inertial force αP and the vertical inertial force α_vP.

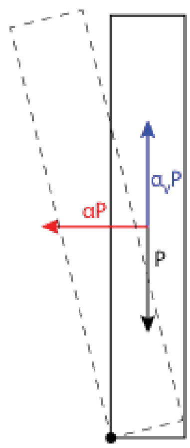


Figure 6. Simple overturning of masonry wall with horizontal and vertical inertial forces

Both inertial forces are proportional to the self-weight. The ratio between the multiplier of vertical inertial force α_v and the multiplier of horizontal inertial force α assumes different values depending on whether the element is considered isolated or resting on the ground (rigid system) or it is located at a certain elevation on the building (deformable system):

$$\frac{\alpha_V}{\alpha} = \frac{k \cdot S_{ez}(0)}{S_{ex}(0)} \qquad \text{in case of rigid system}$$

$$\frac{\alpha_V}{\alpha} = \frac{k \cdot S_{ez}(0)}{S_{ex}(T_1) \cdot \Psi(Z) \cdot \gamma} \qquad \text{in case of deformable system}$$

where: $S_{ex}(T)$ and $S_{ez}(T)$ are the values of the elastic response spectra of horizontal and vertical acceleration for period T; T_1 is the fundamental period of vibration of the whole structure in the horizontal direction; $\Psi(Z)$ is the correspondent mode of vibration normalized so to be 1 at the top of the building; γ is the correspondent modal participation factor; Z is the elevation of the rotation axis; k is the combination factor of the vertical component of the seismic action. k may be taken equal to 0.3 considering the indication given in [8] (4.3.3.5.2), or equal to 1.0 considering that the maximum horizontal and vertical accelerations may occur simultaneously.

Note that in both expression the multiplier of vertical inertial forces is considered proportional to $S_{ez}(0)$, the elastic response spectrum of vertical acceleration for T=0. This because in the proposed methodology the multiplier of vertical inertial forces is always considered equal to the one on rigid system.

The collapse multiplier of the horizontal inertial forces α_0 can be calculated applying the principle of virtual works, with the following relations:

$$\begin{gathered} LV1 + \alpha \cdot LV2 \; + \; \alpha_V \cdot LV3 = 0 \\ LV1 + \alpha_0 \cdot LV2 \; + \; \alpha_0 \cdot \frac{\alpha_V}{\alpha} \cdot LV3 = 0 \\ \alpha_0 = \frac{-LV1}{LV2 + \frac{\alpha_V}{\alpha} LV3} \end{gathered} \tag{1}$$

where: LV1, LV2 and LV3 are respectively the virtual works of static forces, horizontal inertial forces and vertical inertial forces obtained considering $\alpha=1$ and $\alpha_V=1$.

As said, the ratio α_V/α varies depending on whether the system underneath is considered rigid or deformable, but it also varies depending on the Limit State and the level of seismic intensity. Therefore, in the iterative procedure for the calculation of the capacity in terms of PGA the collapse multiplier must be always recalculated based on the current ratio α_V/α. However, given the fact that virtual works LV1, LV2, LV3 remain constant during the iterations, the calculation of the collapse multiplier is not demanding in terms of computational effort.

4 PUSHOVER ANALYSIS

In Pushover analysis the seismic capacity of the structure is described by its behavior under a system of incremental forces which should simulate in the best possible way the inertial forces resulting from the seismic action in the horizontal direction. The capacity of the structure is represented by the capacity curve which is a plot of the total base shear versus the displacement of the control point (normally taken as the center of mass of the roof). In order to assess the vulnerability of the structure several pushover curves must be elaborated for different directions of the seismic action, distributions of lateral forces, effects of accidental eccentricity (modelled as additional twisting moments) and different control points.

Each pushover curve is elaborated through an incremental non-linear procedure performing a series of linear static analyses and keeping the model constantly updated in order to account for the stiffness reduction of the elements which enter the plastic range or reach collapse. According to the current Standards [7], masonry elements are modelled with an elastic-plastic bilinear behavior where the end of the elastic branch is determined by the minimum resistance in terms of different failure mechanisms (bending or shear) and the ultimate displacement is defined through a limit drift (ultimate chord rotation at the two ends of the elements). At each step of the incremental procedure, safety verifications are applied to the elements and whenever the internal actions overcome the resistance, or the deformations overcome the limits, the model is updated accordingly.

Once the pushover curve has been elaborated, each limit state may be associated to a specific point of the curve finding the related capacity in terms of displacement. The target displacement is then defined based on the displacement demand of an equivalent single-degree-of-freedom system derived from the elastic response spectrum. Knowing both capacity and demand in terms of displacement the safety verification can be applied by comparing the two. Moreover, considering that the elastic response spectrum varies with the peak ground acceleration, by iteration one can find which is the capacity of the structure in terms of PGA and calculate the seismic risk index ζ_E as the ratio between capacity and demand in terms of PGA.

The objective of this work is to define a methodology in agreement with current Standards and viable for professionals for considering the vertical component of the seismic action in Pushover analysis. Previous studies proposed an improvement of the traditional pushover analysis taking into account the inertial forces caused by the vertical earthquake. The method was validated through nonlinear time-history analysis [3].

The methodology proposed in this work models the vertical seismic component through a field of vertical spectral forces derived from modal response spectrum analysis considering CQC combination and the elastic response spectrum of vertical acceleration. At each step of the incremental analysis safety verifications are applied to each element combining the effects of static and incremental horizontal loading with those arising from the vertical spectral forces. The latter must be considered both upwards and downwards in order to simulate the effects of the vertical excitation, thus, the verifications are applied twice considering each time the most severe effect. The decompression induced by the forces directed upwards results in a reduction of shear and bending moment resistance while the overpressure induced by the forces directed downward may anticipate a compression failure [5].

According to current Standards [6,8] the action effects due to the combination of the seismic components may be computed combining 100% of the effects in one direction with 30% of the effects in the other directions. The application of this combination to the proposed methodology would imply that the effects of the vertical component should be reduced to 30% since the pushover analysis account for 100% of the effects of the horizontal component. However, for the purposes of this work, the effects of the vertical component will not be reduced since the analysis of the accelerograms of recent earthquakes showed numerous impulses where the maximum ground accelerations in the three directions occur simultaneously.

4.1 Ultimate drift of masonry piers

As stated previously, the shear-displacement behavior of masonry elements is considered bilinear elastic-plastic and the ultimate displacement is defined as a limit drift, that is a limit in terms of chord rotation at the ends of the element. According to current Standards [7] the limit drift for unreinforced masonry piers at the ultimate limit state may be taken as:

$$\delta_u = 0.0125 \cdot (1-\nu) \geq 0.01, where\ \nu = \frac{\sigma_0}{f_d} \qquad \text{in case of bending failure}$$

$$\delta_u = 0.005 \qquad \text{in case of shear failure}$$

Thus, the current Standards, in case of bending failure consider that the ultimate drift may decrease for high values of compression but in case of shear failure and in any case for low compression they provide a fixed value of the limit.

It is understood that the vertical seismic component leads to a drop of resistance with respect to bending and shear mechanisms [4], it shall though be investigated whether it affects also the displacement capacity (ultimate drift) of the elements. Works based on the observation of the damages caused by recent earthquakes showed that the vertical seismic excitation leads to loss of ductility in masonry elements [1]. Since literature does not provide specific experimental evidence, in the proposed methodology the provisions of the current Standards in terms of ultimate drift have been integrated assuming that, applying the vertical seismic component, ductility remains constant.

Figure 7 shows in dashed line the shear-drift behavior of a masonry pier according to current Standards. When the pier reaches the ultimate resistance (F_u; δ_e) it continues to sustain the same action until it reaches the ultimate drift δ_u. The ductility of the pier is defined as the ratio between the ultimate drift and the drift at the end of the elastic branch:

$$\mu = \frac{\delta_u}{\delta_e} \tag{2}$$

The solid line represents the behavior of the same piers under the effects of the vertical component according to the proposed methodology. This time the ultimate resistance drops to F_{uV} which correspond to the drift δ_{eV}. The ultimate drift δ_{uV} is obtained assuming that the ductility of the elements remains constant. Therefore:

$$\delta_{uV} = \delta_{eV} \cdot \mu \tag{3}$$

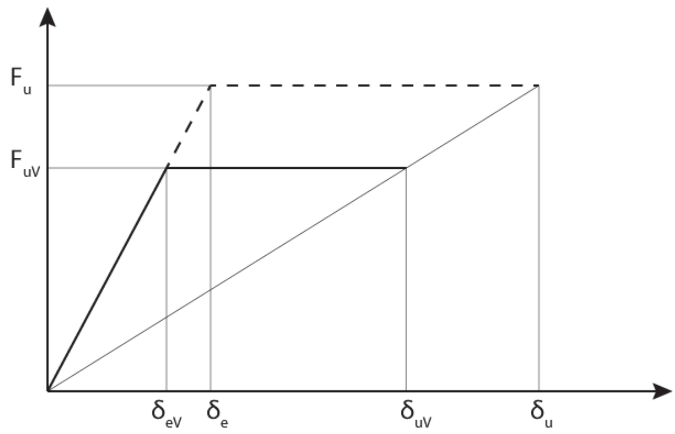


Figure 7. Reduction of ultimate drift due to vertical seismic effects

In this way, every time a masonry pier reaches the ultimate resistance, the ultimate drift is set accordingly based on the value provided by the Standards and reduced in order to account for the effects of the vertical seismic component.

5 CASE STUDY

In order to evaluate the negative effects of the vertical seismic component, the proposed methodology was applied to the case study of a two-story rural building consisting of rubble stone masonry. Pushover analyses were performed with and without the vertical seismic component considering different levels of masonry quality and several levels of seismic action.

The building model shown in Figure 8 features a rectangular floor plan of 13.90x7.40 m with a constant wall thickness of 40 cm. The height of the ground floor is 3.60 m while the first floor with gable roof features eaves height of 3.10 m and ridge height of 3.60 m Openings are not aligned among the two storeys apart from the ones on the west elevation of the building. Floor and roof consist of timber beams topped with 5 cm concrete slab well connected to the perimetral walls.

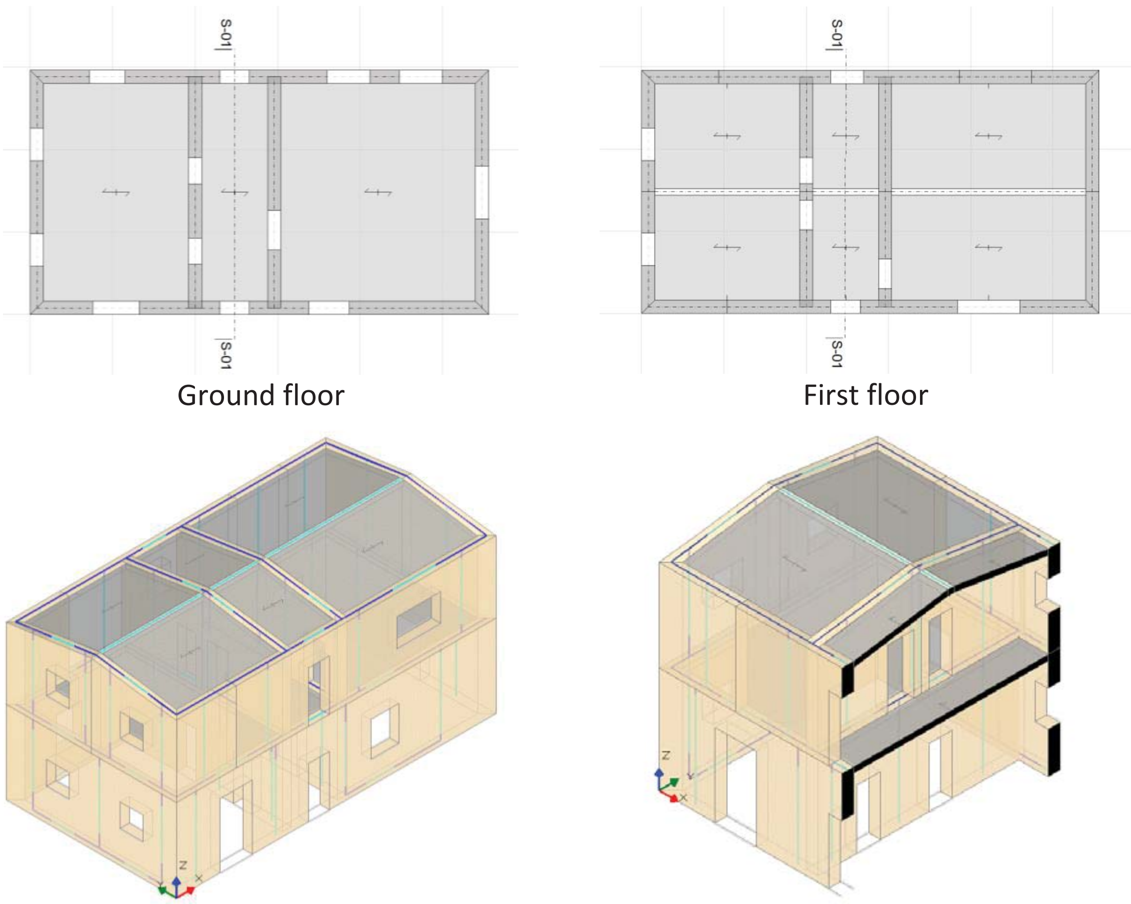


Figure 8. Floor plans, axonometric view and 3D section of the building

For the purposes of this work, modelling and analyses of the building were performed using the commercial software Aedes.PCM [9] which implements the proposed methodology. The building was modelled according to the equivalent frame method where the structure is discretized in a set of masonry panels (piers and spandrels) connected through rigid links. However, the proposed methodology applies to any modelling strategy and the analyses may be carried out with any software able to perform modal and pushover analysis of masonry buildings.

The main characteristics of the model considered in this work are as follows.

- Masonry piers are modelled with a trilinear in-plane behavior consisting of two elastic branches and one perfectly plastic. When tensile stresses appear in the cross-section shear and stiffness are reduced by 50% and the element enters the second elastic branch. Then, when the element reaches ultimate resistance in terms of shear or flexure mechanisms plastic hinges are introduced and the element enters the plastic branch. In this last branch of the shear-displacement behavior, shear and bending moment remain constant until the element reaches ultimate deformation.
- In order to simplify the analysis, masonry spandrels are considered able to couple masonry piers only with respect to horizontal translation, thus the rotations are released at both their ends
- Restraints. Joints at the foundation of the building are assumed fully fixed.
- Vertical loading. The intermediate slabs carry a dead load of 2.45 kN/m^2 and a live load of 2.00 kN/m^2 (cat. A). The roof carries a dead load of 2.45 kN/m^2 and a live load of 1.20 kN/m^2 (snow). Since the timber beams span along the longitudinal direction of the

building, all the slabs distribute 90% of the loads to the transversal walls and 10% to the longitudinal ones.
- Verifications. The in-plane resistance of masonry piers is governed by the following mechanisms: flexure, diagonal shear. Considering the relevant thickness of the piers (40 cm) their transversal stiffness is also accounted in the analysis and out-of-plane flexure verification applied at both their ends.

The walls of the building consist of rubble stone masonry. In order to evaluate the effects of the vertical seismic component, six different levels of masonry quality were considered in the analysis. The mechanical properties associated to each level are based on the reference values for rubble stone masonry provided by the Italian Standards [7]. Assuming a knowledge level KL2, the reference values of resistance and moduli of elasticity were taken as the mean values of the provided range and a confidence factor CF = 1.2 was accounted in the analysis. Corrective coefficients related to specific characteristics or strengthening measures (also provided by the Standards) were applied to create a scale of quality levels as specified in Table 1.

Level	**Description**	**E**	**G**	**f_m**	**τ_0**
A1	core of poor quality	783	261	1.35	0.0225
A2	standard condition	870	290	1.50	0.0250
A3	mortar of good quality	1305	435	2.25	0.0375
A4	mortar of good quality and lacing courses	1305	435	2.92	0.0487
A5	reinforced mortar coating	2175	725	3.75	0.0625
A6	best possible interventions	3045	1015	5.25	0.0875

Table 1. Mechanical properties of six different level of masonry quality: modulus of elasticity (E), shear modulus (G), mean compressive strength (fm), initial shear strength under zero compression (τ0). Values in N/mm^2

The building is assumed located in Perugia (Italy). The Italian Standards [6] provide, for each location of the Italian territory and for different return periods T_R, the reference parameters of the seismic action and the methods to consider stratigraphic and topographic amplifications. The parameters of the elastic response spectra of horizontal and vertical accelerations with respect to the Ultimate State of Severe Damage (T_R = 475) are given in Table 2.

		Horizontal	**Vertical**	
Ground acceleration	**a_g**	0.186	0.186	g
Soil factor	**S**	1.200	1.000	
Maximum amplification factor	**F**	2.425	1.412	
Periods	**T_B**	0.103	0.050	s
	T_C	0.310	0.150	s
	T_D	2.344	1.000	s

Table 2. Seismic parameter for T_R = 475 years

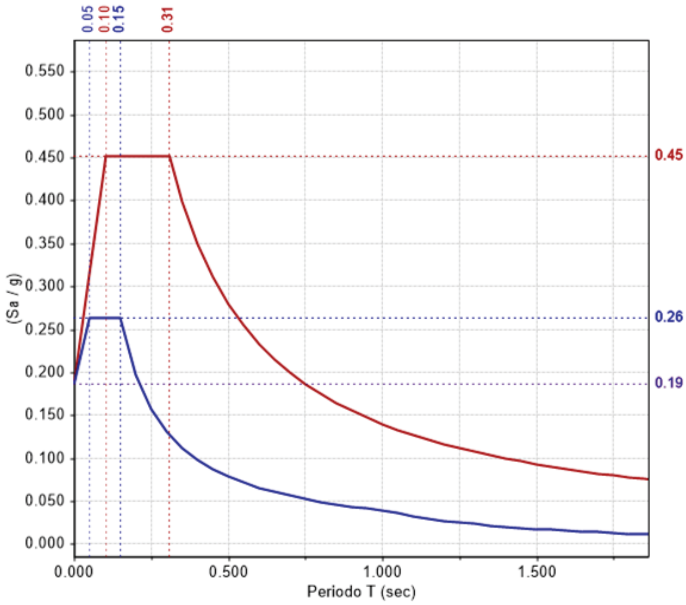


Figure 9. Elastic response spectra of horizontal (red) and vertical (blue) acceleration for T_R = 475

One of the aims of this work is to evaluate how the effects of the vertical seismic component vary considering increasing levels of seismic intensity. Therefore, the analyses were performed considering 13 different levels of seismic action. The parameters of the response spectra associated to each level were obtained from the reference values provided by the Standards considering increasing values of horizontal and vertical ground accelerations assuming that all the other parameters of the spectra remain constant.

Table 3 provides the values of ground acceleration for the 13 different levels of intensity considered in the analysis. The values are given with respect to a return period T_R=475 years (the parameters associated to other return periods necessary for the calculation of the seismic risk index are updated accordingly).

Level	a_g
1	0.186 g
2	0.250 g
3	0.300 g
4	0.350 g
5	0.400 g
6	0.450 g
7	0.500 g
8	0.550 g
9	0.600 g
10	0.650 g
11	0.700 g
12	0.750 g
13	0.800 g

Table 3. Horizontal and vertical ground acceleration for 13 different levels of seismic action with respect to T_R = 475 years.

5.1 Modal Analysis

Modal analysis was performed considering the diaphragm actions provided by the rigid levels. The results of the analysis are given in Table 4, while Figure 10-12 show the deformed shapes associated to the fundamental mode of vibration in X Y and Z direction.

Mode	Eigenvalue $(rad/sec)^2$	Period (sec)	Participating mass ratio (%)			Participating mass ratio (progressive total %)		
			X	Y	Z	X	Y	Z
1	4.13E+02	0.309	0.356	76.240	0.000	0.356	76.240	0.000
2	4.36E+02	0.301	92.813	0.016	0.010	93.169	76.256	0.010
3	7.64E+02	0.227	1.086	15.680	0.000	94.254	91.936	0.011
4	3.65E+03	0.104	0.000	7.619	0.003	94.254	99.555	0.014
5	4.43E+03	0.094	5.482	0.000	0.032	99.736	99.555	0.046
6	6.10E+03	0.080	0.083	0.066	0.001	99.819	99.621	0.047
7	1.12E+04	0.059	0.001	0.004	32.427	99.820	99.625	32.474
8	1.14E+04	0.059	0.033	0.007	2.502	99.853	99.632	34.976
9	1.22E+04	0.057	0.001	0.000	15.378	99.854	99.632	50.354
10	1.23E+04	0.057	0.000	0.000	26.747	99.854	99.632	77.101
11	1.35E+04	0.054	0.000	0.008	8.642	99.854	99.640	85.744
12	1.44E+04	0.052	0.007	0.039	0.678	99.861	99.679	86.422
13	1.51E+04	0.051	0.000	0.021	5.526	99.861	99.700	91.948

Table 4. Modal analysis results

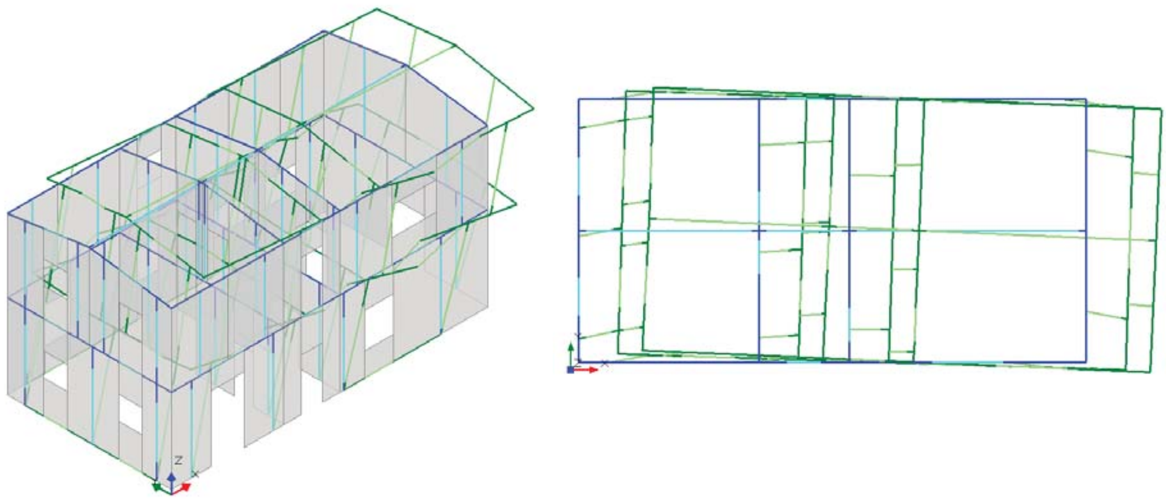


Figure 10. Fundamental mode of vibration in X direction

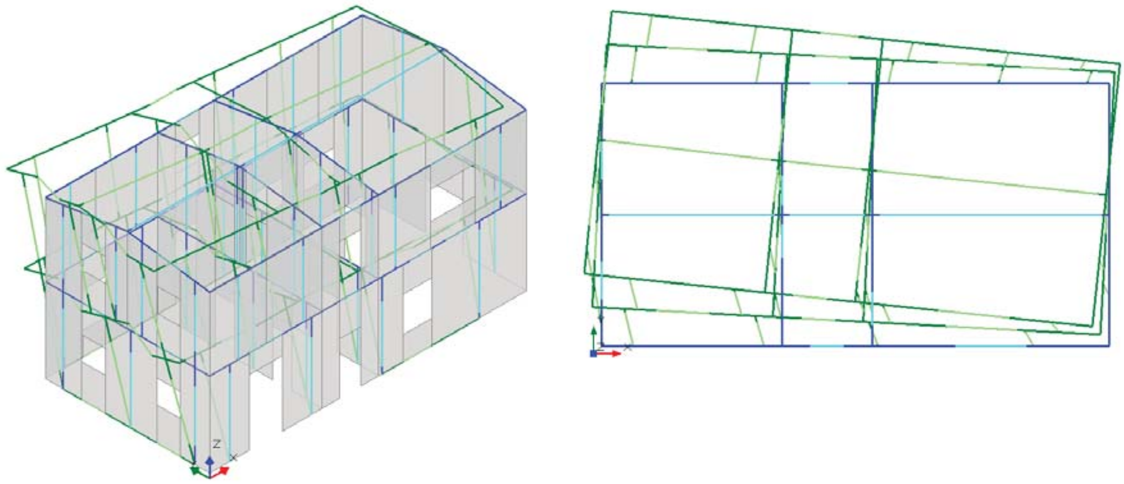


Figure 11. Fundamental mode of vibration in Y direction

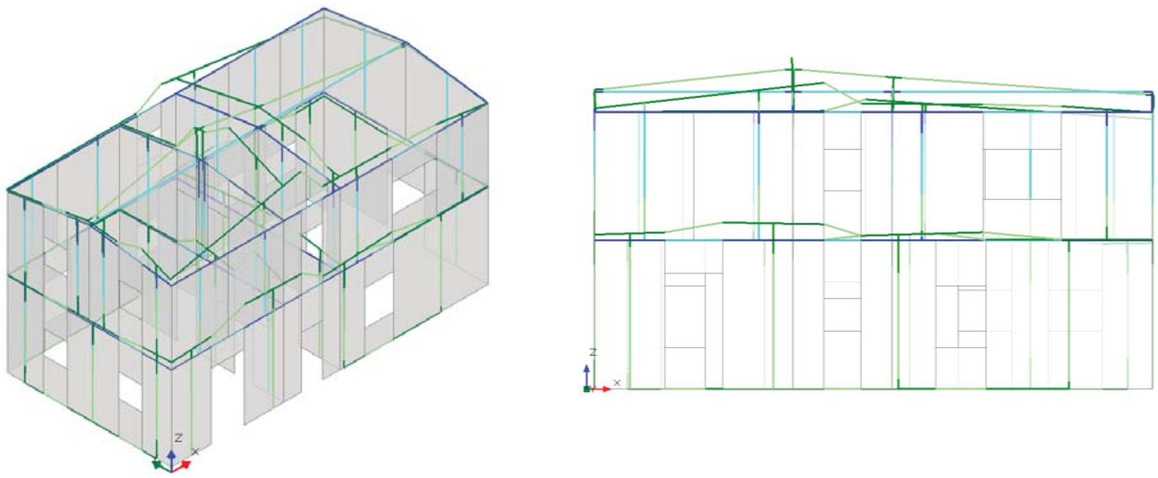


Figure 12. Fundamental mode of vibration in Z direction

It is interesting to observe the high frequencies associated to the vertical modes of vibration. In order to activate 85% of the total mass in each direction and consider all the modes with at least 5% participating mass it was necessary to use all the 13 modes listed in Table 4. For each level of seismic intensity considered in the analysis, the vertical dynamic forces associated to each mode of vibration were calculated using the elastic response spectrum of vertical acceleration and combined through CQC method. The resulting field of forces is represented in Figure 13. The internal actions arising from this field of forces were taken into account at each step of Pushover analysis in the safety verifications of masonry elements.

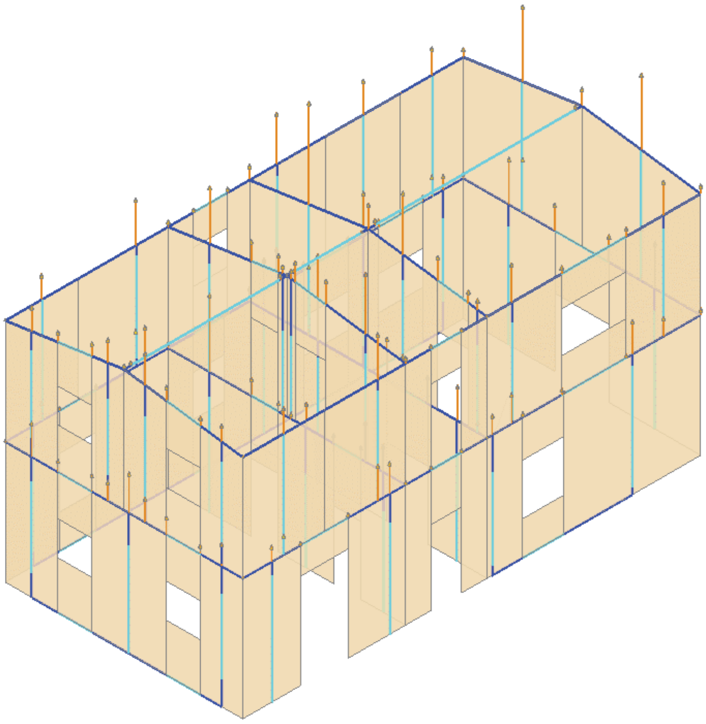


Figure 13. Vertical spectral forces

5.2 Pushover analysis

Pushover analyses were performed for 6 different levels of masonry quality and 13 different levels of seismic action, thus for a total of 78 cases.

For each case the analysis was performed through elaboration of 16 pushover curves differing for:

- distributions of horizontal forces: a linear distribution with forces proportional to masses and elevations and a uniform distribution with forces proportional only to masses
- directions: the positive X and Y direction, respectively the longitudinal and transversal direction of the building
- effects of accidental eccentricity: considering additional twisting moment applied clockwise and counterclockwise
- effects of the vertical seismic component: curves with and without the effects of the vertical forces

The results of the analyses will be compared with respect to the Seismic Risk Index ζ_E, that is the ratio between capacity and demand in terms of PGA. For the purposes of this work the capacity in terms of PGA was calculated assuming that the effects of the vertical seismic component remain constant throughout the iterative procedure. For professional applications a method for considering the variability of the vertical seismic effects in the calculation of the capacity in terms of PGA is provided in §6.1.2.

Figure 14 shows two of the pushover curves elaborated for masonry A6 considering the first level of seismic intensity (a_g = 0.186 g). The curves refer to the analysis in +X direction with linear distribution of horizontal forces and twisting moments applied counterclockwise. The blue line represents the curve without the effects of the vertical seismic component while the red line is the curve obtained considering those effects. The point shown in both curves represents the capacity with respect to the Ultimate Limit State of Severe Damage while the dashed line represents the relevant displacement demand.

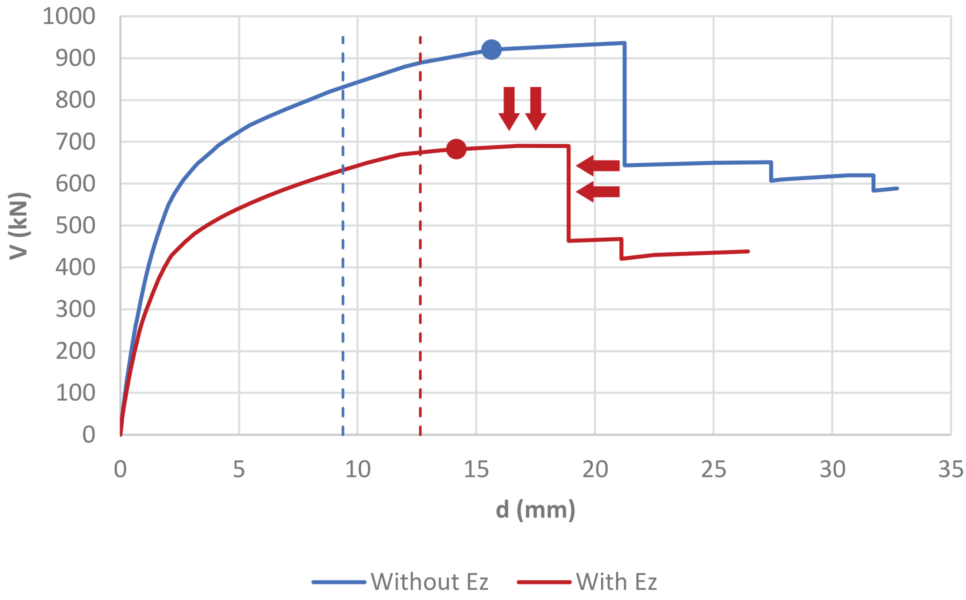


Figure 14. Pushover curve in +X direction with linear distribution of horizontal forces

It is evident that the vertical seismic component produces a reduction of global resistance and displacement capacity. Specifically, the maximum base shear drops by 26% while the displacement capacity at the ULS drops by 11%. At the same time the displacement demand increases by 34%. Finally, the Seismic Risk Index ζ_E, that is the ratio between capacity and demand in terms of PGA, drops from 1.381 to 1.081 with a reduction of 22%.

Figure 15 shows the configuration of the structure at the last step of the pushover curve with the effects of the vertical seismic component. The different colors represent the state of the piers: elastic (green), partially plastic (yellow), plastic (orange), collapsed (red).

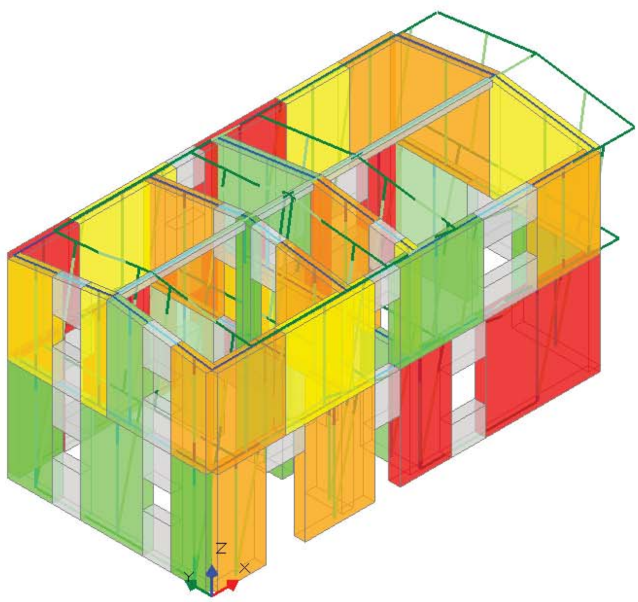
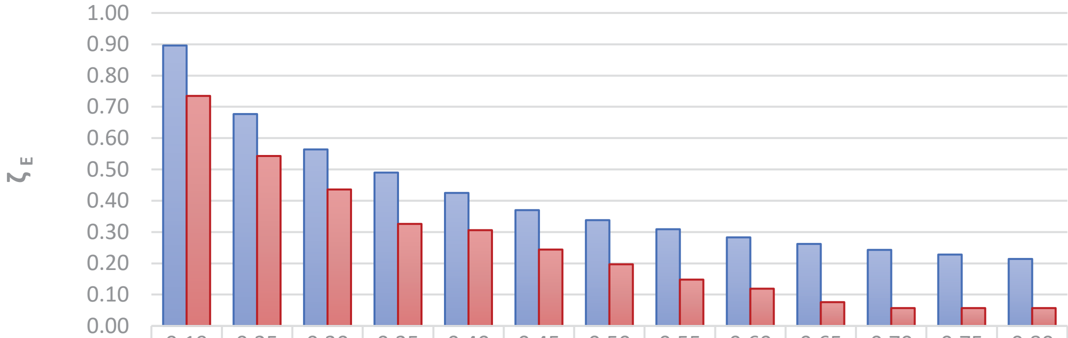


Figure 15. Displacements and state of the elements at the last step of pushover curve

The following consideration are based on the minimum values of the Seismic Risk Index arising from the elaboration of all the pushover curves for each of the 78 cases considered.

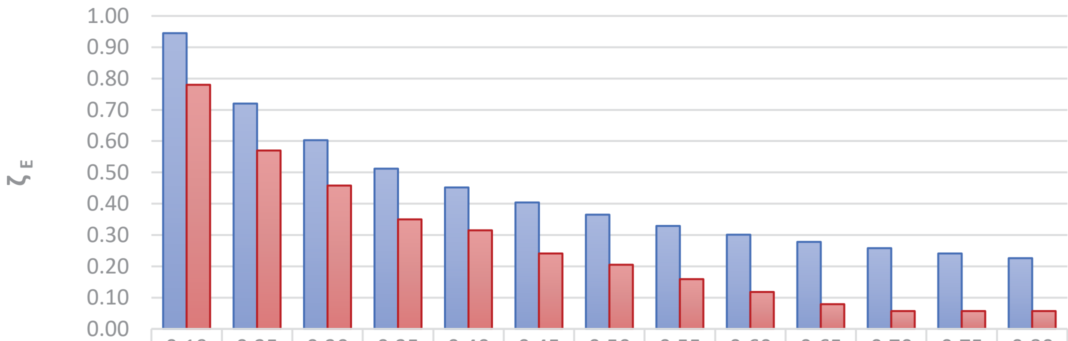
Figures 16-21 show for each levels of masonry quality the variation of the Seismic Risk Index with the level of seismic action represented by the vertical ground acceleration a_{gV}. The diagrams show with blue bars the Seismic Risk Index obtained without the vertical seismic component and with red bars the value obtained considering those effects.



	0.19	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80
Without Ez	0.896	0.677	0.564	0.490	0.425	0.370	0.338	0.309	0.283	0.262	0.243	0.228	0.214
With Ez	0.735	0.543	0.436	0.326	0.306	0.244	0.197	0.148	0.119	0.076	0.057	0.057	0.057

agV (g)

Figure 16. Variation of Seismic Risk Index for masonry A1



	0.19	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80
Without Ez	0.945	0.720	0.603	0.512	0.452	0.404	0.365	0.329	0.301	0.278	0.258	0.241	0.226
With Ez	0.780	0.570	0.458	0.350	0.315	0.241	0.205	0.159	0.118	0.079	0.057	0.057	0.057

agV (g)

Figure 17. Variation of Seismic Risk Index for masonry A2

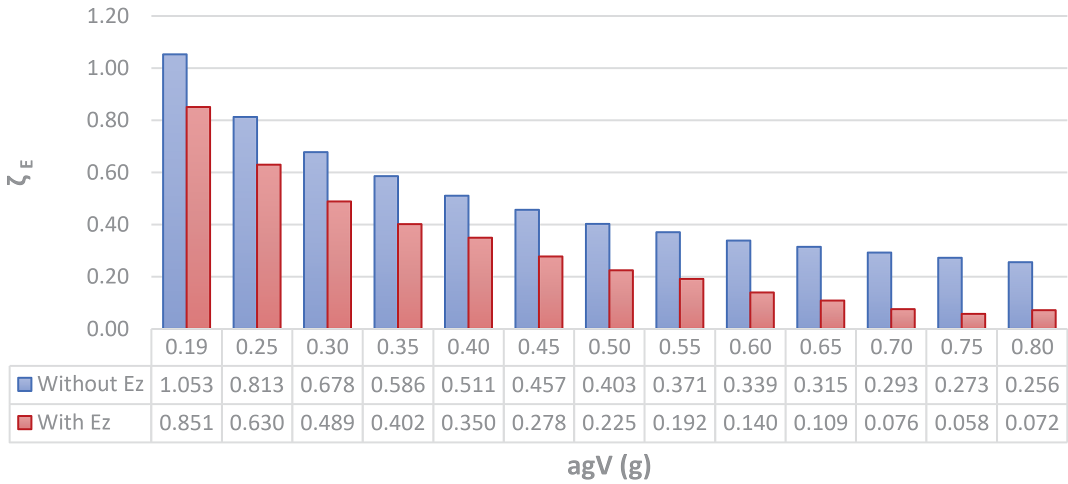


Figure 18. Variation of Seismic Risk Index for masonry A3

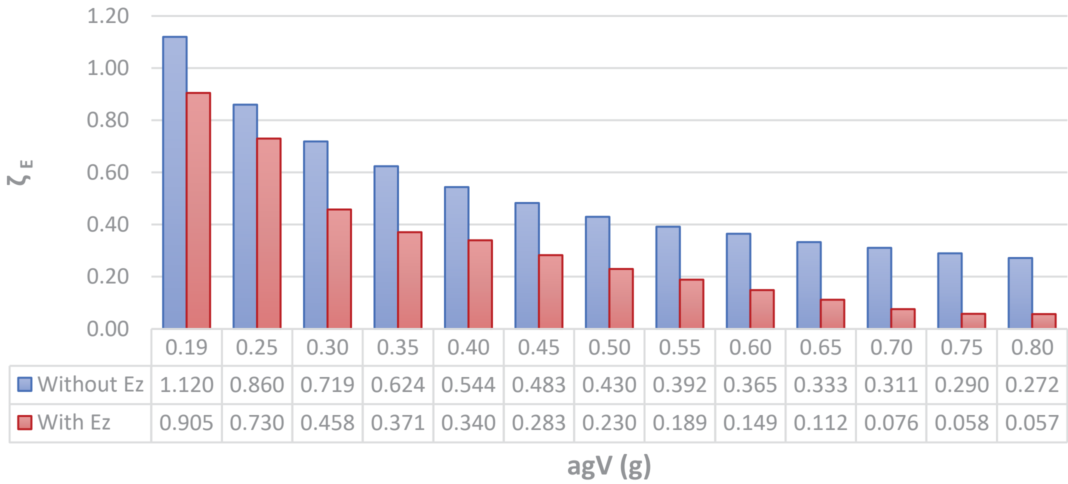


Figure 19. Variation of Seismic Risk Index for masonry A4

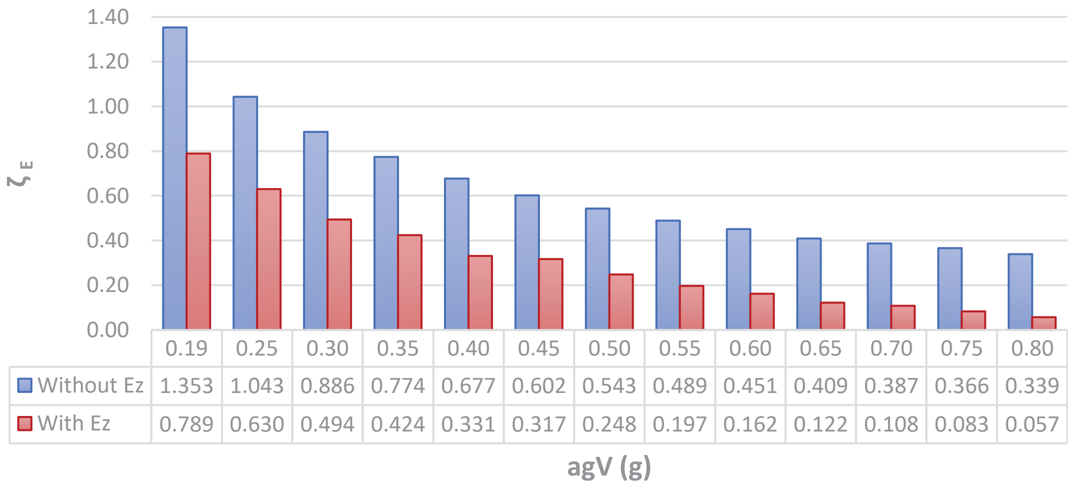
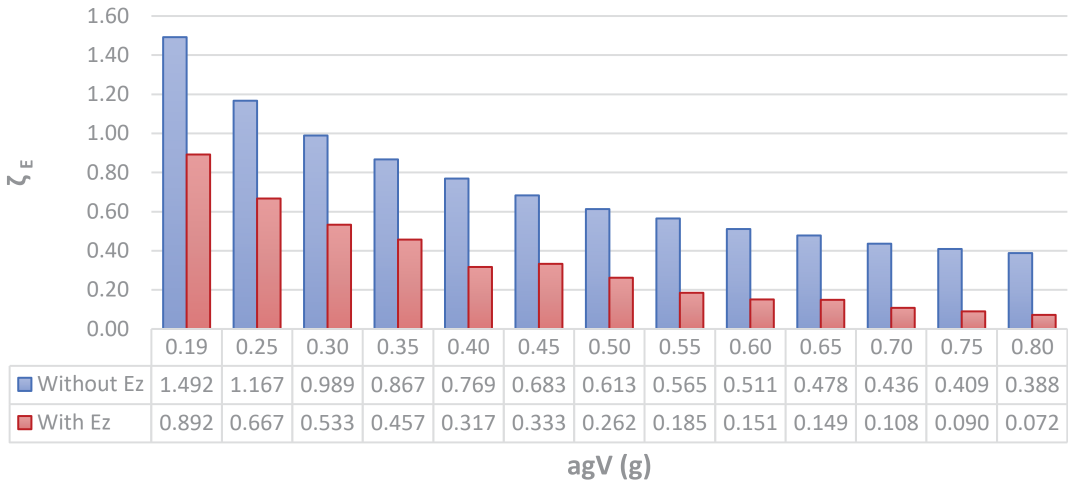


Figure 20. Variation of Seismic Risk Index for masonry A5



	0.19	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80
Without Ez	1.492	1.167	0.989	0.867	0.769	0.683	0.613	0.565	0.511	0.478	0.436	0.409	0.388
With Ez	0.892	0.667	0.533	0.457	0.317	0.333	0.262	0.185	0.151	0.149	0.108	0.090	0.072

Figure 21. Variation of Seismic Risk Index for masonry A6

The Seismic Risk Indexes decrease as the ground acceleration increases and the application of the vertical seismic component always leads to lower indexes. Moreover, as the ground acceleration increases also the gap between the two cases increases.

This behavior is more evident in Figure 22 that shows for each levels of masonry quality the reduction of the Seismic Risk Index caused by the application of the vertical seismic component. As expected, the reduction increases with the value of the ground acceleration a_{gV}. However, we also notice that the reduction is higher for masonries of better quality, meaning that the vertical seismic component tends to reduce the benefic effects of eventual strengthening interventions.

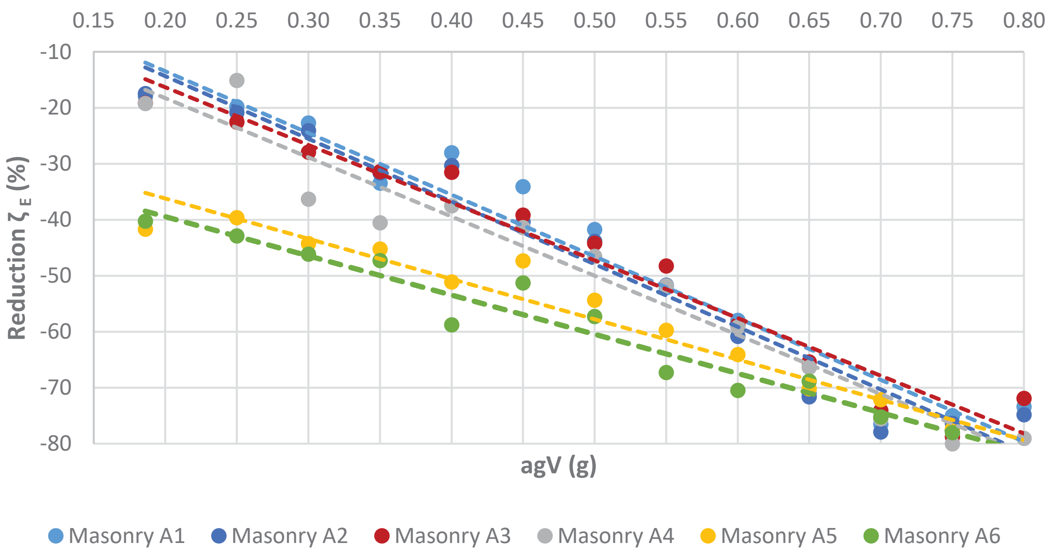


Figure 22. Reduction of the Seismic Risk Index caused by the vertical seismic effects

The diagrams in Figures 16-21 may be assembled in the following diagrams showing the variation of the Seismic Risk Index with the level of seismic intensity and the level of masonry quality in the cases with and without the vertical seismic component.

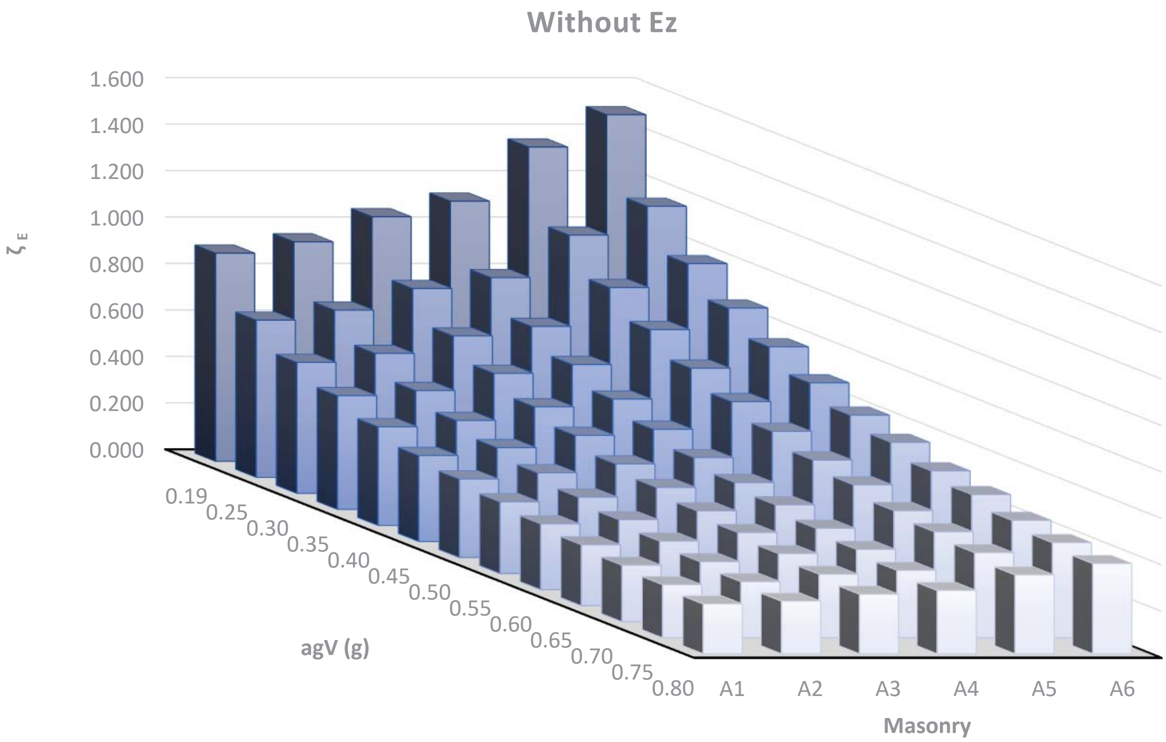


Figure 23. Variation of the Seismic Risk Index with levels of masonry quality and seismic action ignoring the vertical seismic effects

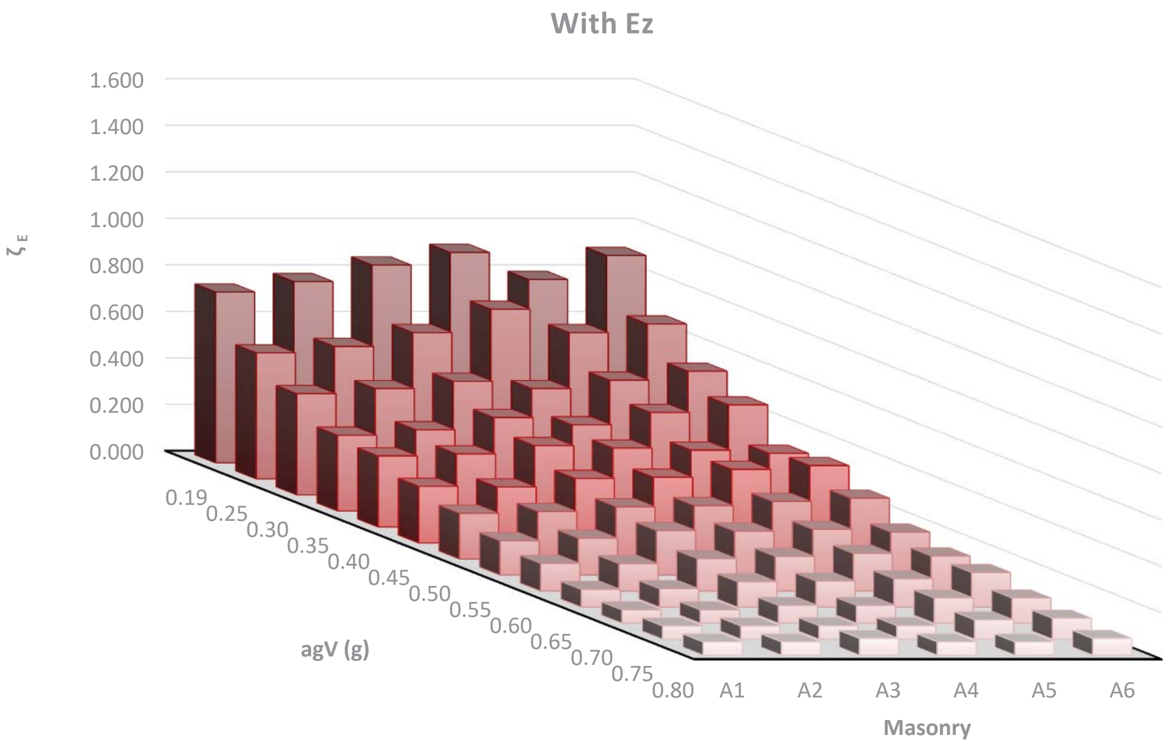
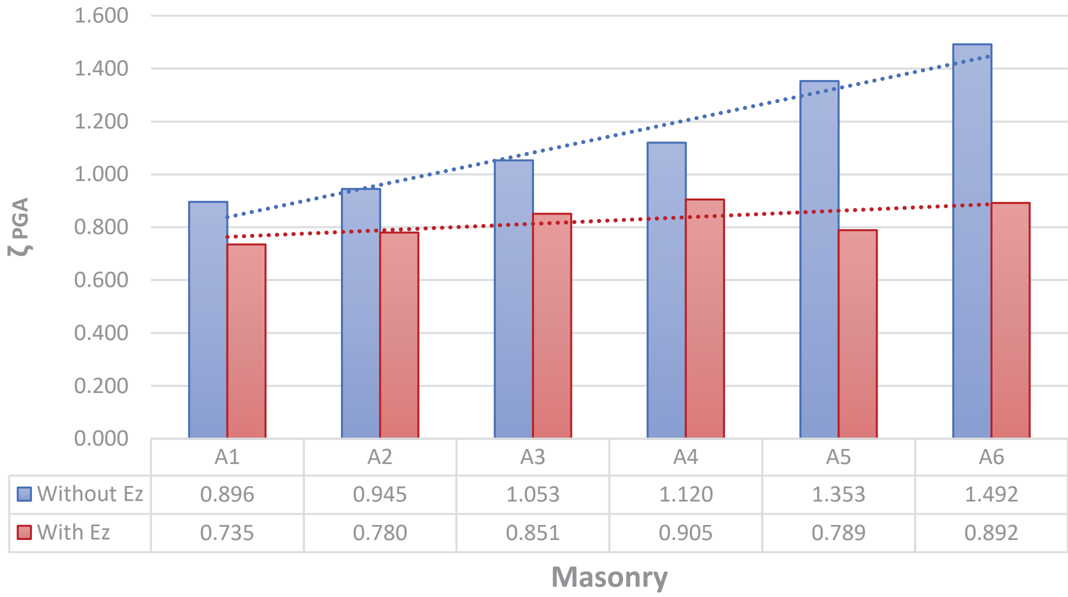


Figure 24. Variation of the Seismic Risk Index with levels of masonry quality and seismic action considering the vertical seismic effects

When the effects of the vertical seismic component are ignored (Figure 23) the Seismic Risk Index decrease as the ground acceleration increase while it increases with the level of masonry quality. Considering the effects of the vertical seismic component (Figure 24) the improvement obtained with masonries of better quality is still evident but not so sharp like in the other case.

Figure 25 shows the variation of the Seismic Risk Index with the quality of masonry for the first level of seismic action (a_{gv} = 0.186g). In both cases the indexes tend to increase with the quality of masonry. However, considering the vertical seismic effects the increment is more moderate.



	A1	A2	A3	A4	A5	A6
Without Ez	0.896	0.945	1.053	1.120	1.353	1.492
With Ez	0.735	0.780	0.851	0.905	0.789	0.892

Figure 25. Variation of the Seismic Risk Index with the level of masonry quality for the first level of seismic action (a_{gV}=0.186 g)

6 PROFESSIONAL APPLICATION

The proposed methodology for considering the vertical seismic component in the assessment of masonry buildings may be applied to real cases taking into accounts the aspects highlighted in §6.1. The safety assessment of a real masonry building with the application of the proposed methodology is presented in §6.2.

6.1 Specific aspects of the methodology

6.1.1 Influence of compression and decompression cycles on resistance of masonry piers

In pushover analysis the progressive deterioration caused by horizontal and vertical components of the seismic action must be addressed in order to adequately represent the real masonry behavior. During seismic excitation numerous cycles of compression and decompression induced by the vertical seismic component may determine irreversible deterioration and decohesion of mortar joints. The poorer the quality of mortar, the more important this phenomenon is.

The phenomenon could be represented through reduction of the compressed part of the cross section caused by decompression. This leads to reduction of bending and shear resistance provided that the sliding shear mechanism is considered in the model. In irregular masonries this is often not the case due to the absence of a clearly horizontal mortar bed that could activate this failure mechanism. In the proposed methodology, in order to account for decohesion of the mortar joints caused by the vertical cycles, verification with respect to sliding shear mechanism is applied also for irregular masonries and the value of initial shear strength under zero compressive stress is assumed equal to minimum value provided by the current Standards, that is f_{v0}=0.07 N/mm^2.

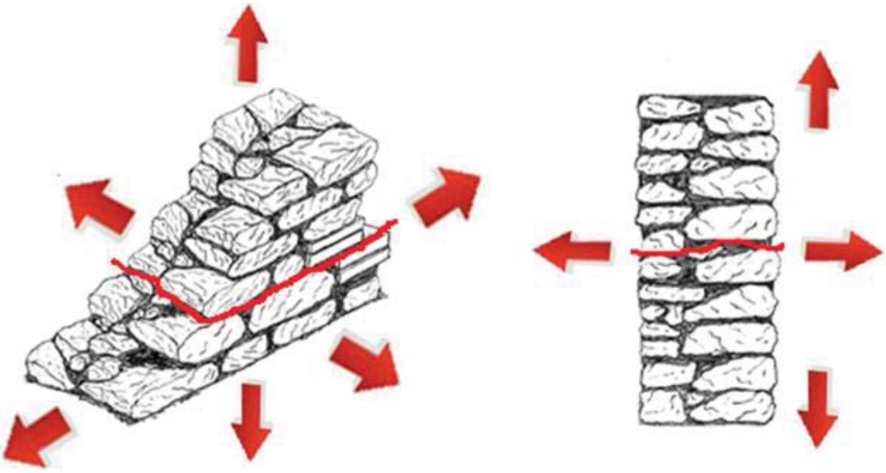


Figure 26. Possible sliding shear failure in uncut stone masonry caused by compression and decompression cycles due to vertical seismic action

6.1.2 Capacity of the structure in terms of PGA

Once pushover curve has been elaborated it allows to perform a safety verification of the structure comparing the capacity and the seismic demand in terms of displacement. Considering that the displacement demand varies with the value of PGA, the pushover curve allows to find the capacity of the structure in terms of PGA and calculate the Seismic Risk Index ζ_E as the ratio between capacity and demand.

In pushover curves that ignore the vertical seismic component the capacity in terms of PGA can easily be found through an iterative procedure applying the displacement verification for different values of the demand. During this iteration the pushover curve remains the same because it is an intrinsic property of the structure and does not depend on the seismic demand.

This is not the case in pushover curves elaborated considering the vertical seismic component, given that the vertical spectral forces are calculated based on the elastic response spectrum of vertical acceleration. Therefore, the pushover curve depends on the seismic demand, as a result the iterations to find the capacity in terms of PGA would require re-elaboration of the pushover curve at each step for different values of the seismic demand. This procedure would be very demanding in terms of computational effort and hardly feasible in professional applications. For this reason, this work proposes an alternative manner for individuating the capacity in terms of PGA based on a linear interpolation between the results obtained with and without the vertical seismic component.

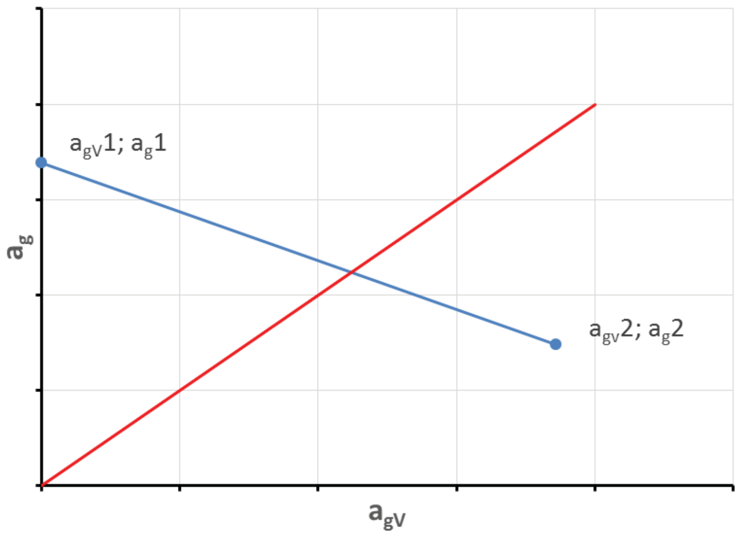


Figure 27. Capacity in terms of PGA obtained through interpolation

Consider the graph in Figure 27, where the a_{gV} axis represents vertical ground acceleration and the a_g axis represents horizontal ground acceleration. Point 1 (a_{gv1}; a_{g1}) represents the capacity of the structure in terms of horizontal ground acceleration resulting from a curve without the vertical seismic component ($a_{gv1} = 0$). Point 2 (a_{gv2}; a_{g2}), instead, represents the capacity of the structure resulting from a curve where the applied vertical ground acceleration is equal to the seismic demand (in this case the capacity in terms of horizontal ground acceleration a_{g2} has been calculated assuming that the capacity curve remains the same throughout the iterative procedure). The blue line passing by point 1 and point 2 represents an estimation of how the capacity in terms of horizontal acceleration varies with the vertical acceleration applied in the analysis. The red line represents the ratio between horizontal and vertical acceleration, which, according to current Standards, is assumed constant for each level of seismic intensity. The intersection between the blue line and red line individuates the real capacity of the structure in terms of horizontal ground acceleration when the effects of the vertical component are considered. In this way the Seismic Risk Index in terms of PGA can be calculated and the procedure can be applied for each limit state.

6.2 Seismic assessment of a real masonry building

The objective of this study is the seismic assessment of a real masonry building located near Macerata, Italy. The assessment will be carried out through pushover analysis applying the proposed methodology for taking into account the vertical seismic effects. First the building will be assessed in its current state, then the effects of possible strengthening interventions will be evaluated. The two-story building shown in Figure 28 features a hipped roof and rectangular floor plan of 26x16 m.

Walls are made of uncut stone masonry and are well connected with each other. Perimetral walls feature a thickness of 50 cm while internal walls are 30 cm thick. The mechanical parameters of masonry are as follows: f_m=2.00, τ_0 =0.035, E=1230, G=410 (N/mm^2). The achieved knowledge level is KL1, thus a confidence factor CF=1.35 is accounted in the analysis.

Floors consist of a series of littles vaults supported by steel beams while roof is made of timber beams and timber planks. Both floors and roof may be considered non rigid in their plane. Vertical loading is given in Table 5.

	Dead load	Live load
First floor	3.10	2.00 (Cat.A)
Second floor	4.50	2.00 (Cat.A)
Roof	2.25	1.33 (Snow)

Table 5. Vertical loading on floors and roof. Values in kN/m^2

The parameters of the seismic action with respect to the Ultimate State of Severe Damage (T_R = 475) are given Table 6.

		Horizontal	Vertical	
Ground acceleration	a_g	0.223	0.223	g
Soil factor	**S**	1.185	1.000	
Maximum amplification factor	**F**	2.415	1.540	
	T_B	0.147	0.050	s
Periods	T_C	0.442	0.150	s
	T_D	2.492	1.000	s

Table 6. Seismic parameter for TR = 475 years

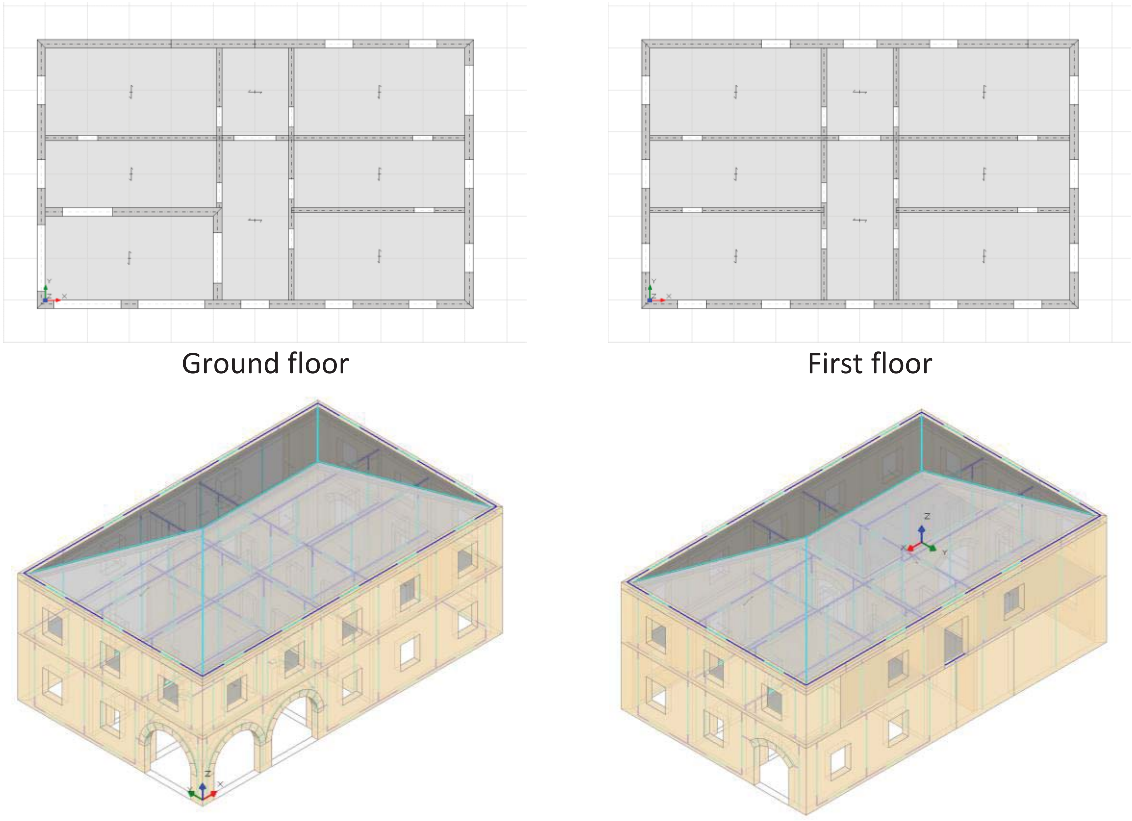


Figure 28. Floor plans and axonometric views of the building

Modelling and analyses of the building were performed using the commercial software Aedes.PCM [9] which implements the proposed methodology. The building was modelled according to the equivalent frame method with the following characteristics:

- Masonry piers are modelled with a trilinear in-plane behavior consisting of two elastic branches and one perfectly plastic. In the second elastic branch stiffness are reduced by 50%.
- Masonry spandrels are considered able to couple masonry piers only with respect to horizontal translation, thus the rotations are released at both their ends
- Restraints. Joints at the foundation of the building are assumed fully fixed.
- Verifications. The in-plane resistance of masonry piers is governed by the following mechanisms: flexure, diagonal shear, sliding shear in case of vertical seismic action. Considering the relevant thickness of the piers their transversal stiffness is also accounted in the analysis and out-of-plane flexure verification applied at both their ends.

The results of modal analysis with respect to the fundamental mode of vibration in X, Y and Z direction are given in Table 7. Figures 29-31 show the correspondent deformed shapes.

Mode	T (sec)	Part. mass ratio (%)
X	0.310	70.1
Y	0.503	86.3
Z	0.085	14.9

Table 7. Fundamental mode of vibrations

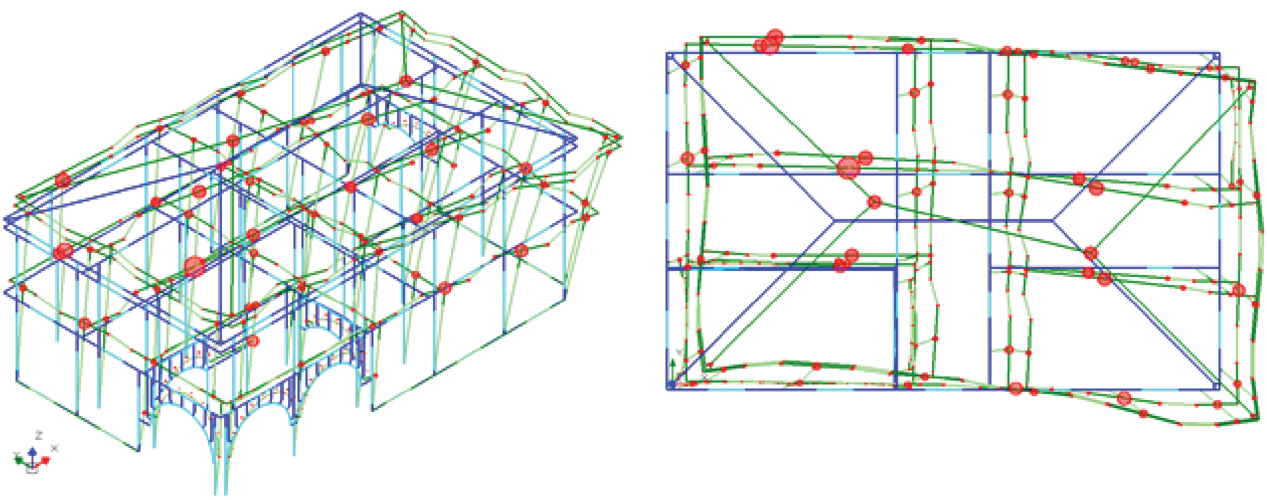


Figure 29. Fundamental mode of vibration in X direction

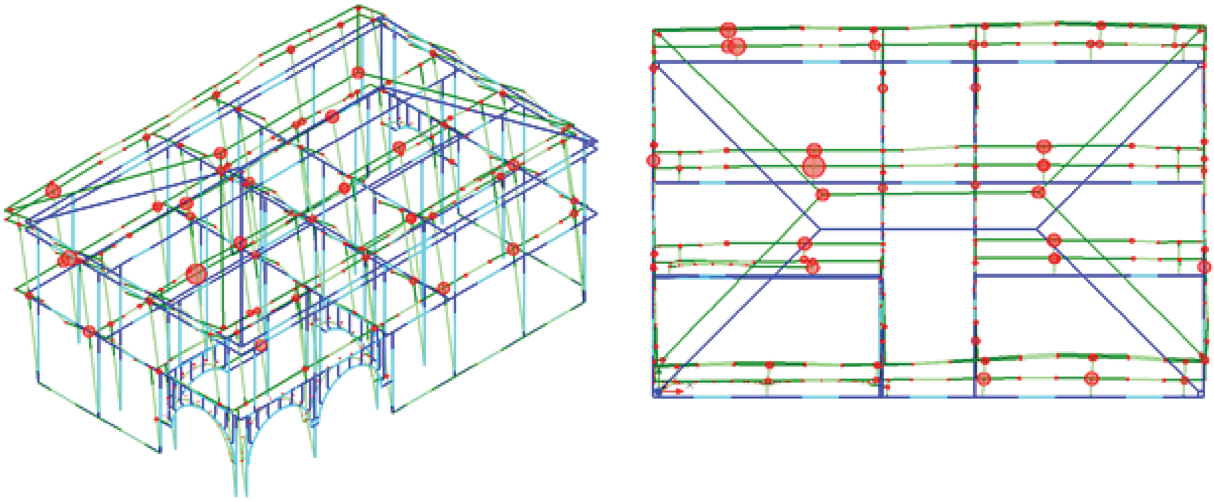


Figure 30. Fundamental mode of vibration in Y direction

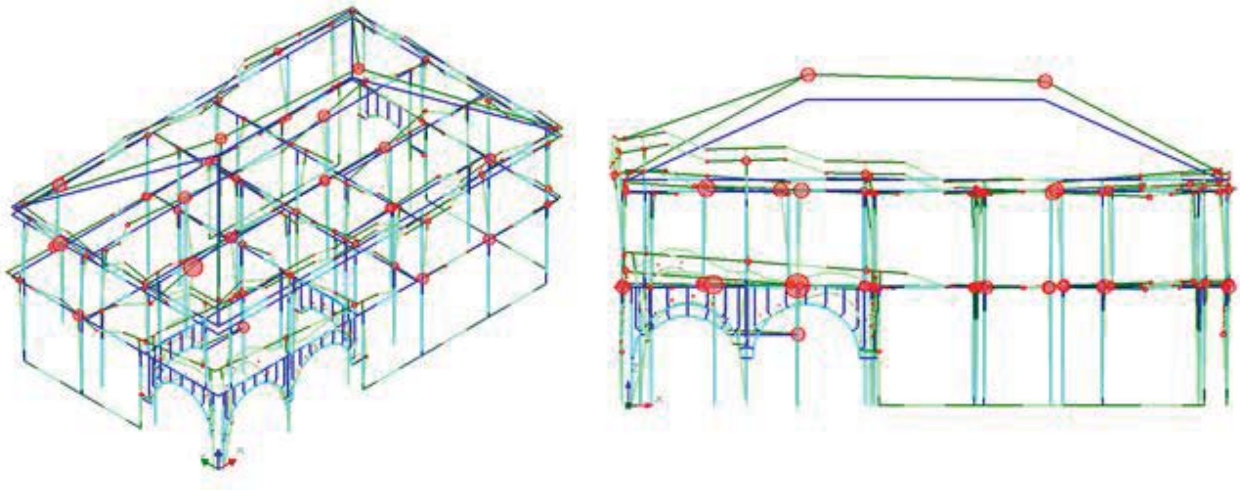


Figure 31. Fundamental mode of vibration in Z direction

Pushover analysis was performed through elaboration of 48 capacity curves differing for:

- distributions of horizontal forces: a linear distribution with forces proportional to masses and elevations and a uniform distribution with forces proportional to masses
- directions: X and Y directions, respectively the longitudinal and transversal direction of the building
- effects of the vertical seismic component: curves with and without the effects of the vertical forces

Figure 32 shows the curves that yielded the minimum values of the seismic risk index in terms of PGA with and without the effects of the vertical seismic effects. They are the curves in the positive and negative Y direction obtained with uniform distribution of the horizontal forces. The arrows highlight the reductions of maximum base shear and displacement capacity at ULS due to the vertical seismic component given in more detail in Table 8.

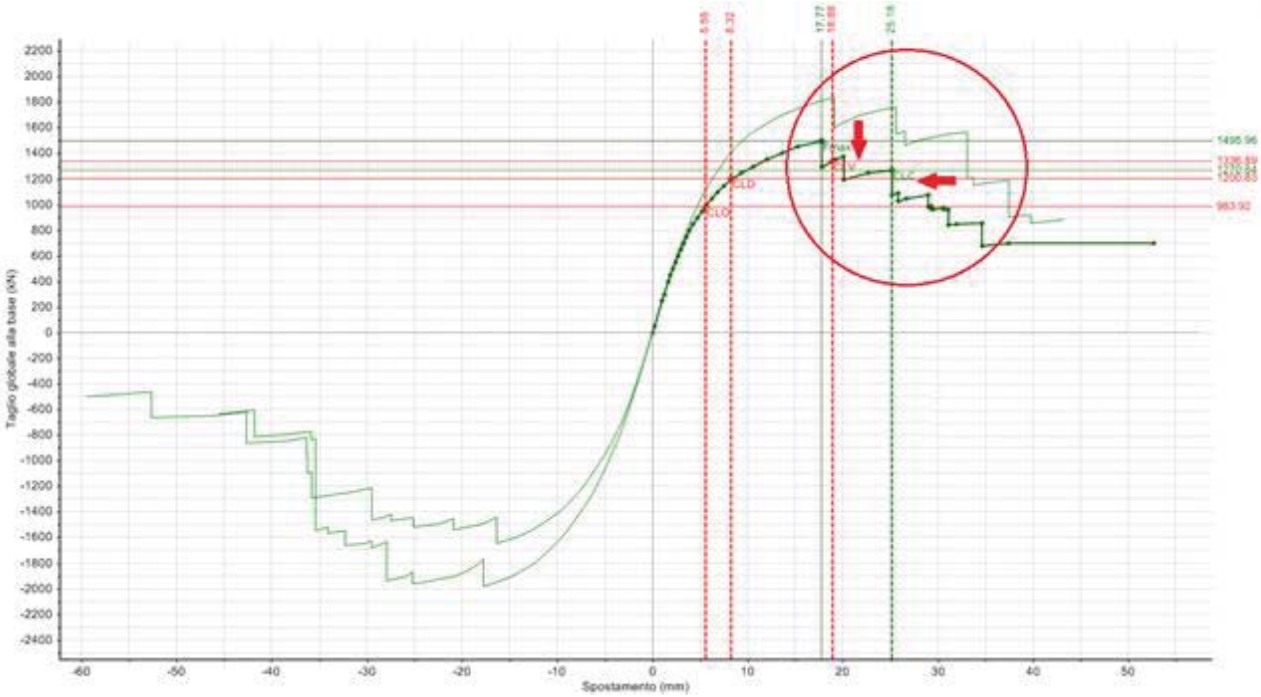


Figure 32. Curve in positive Y direction with uniform distribution and vertical seismic component compared with other curves in Y direction

	Without E_Z	With E_Z	
Maximum base shear (kN)	1837	1496	-19%
Displacement capacity (mm)	24.80	18.88	-24%
Seismic Risk Index ζ_E	0.700	0.609	-13%

Table 8. Reduction of maximum base shear, displacement capacity at ULS and seismic risk index due to vertical seismic component E_Z

In order to improve the seismic capacity of the structure the following strengthening interventions were considered: (a) application of reinforced mortar coating on the perimetral walls and on the internal walls in the transversal direction of the building as shown in Figure 33, (b) stiffening of the floors through lightweight concrete slabs well connected to perimetral walls.

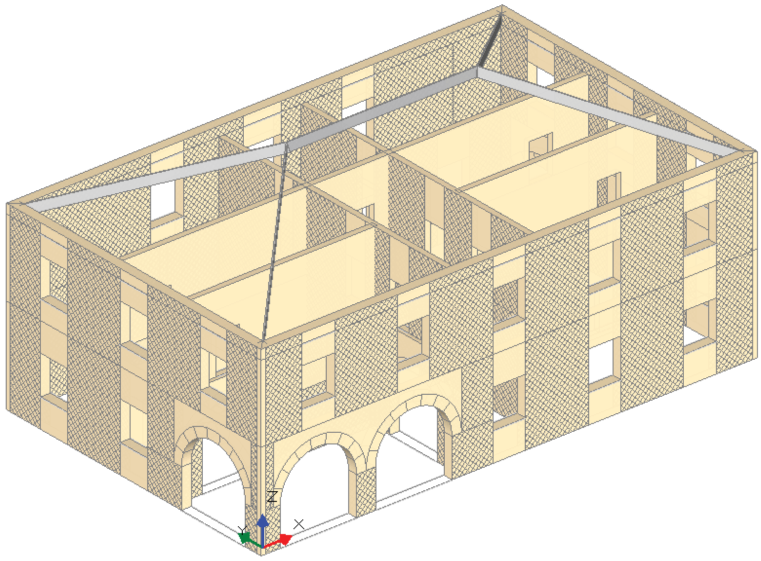


Figure 33. Application of reinforced mortar coating on perimetral and internal walls

The application of reinforced mortar coating was accounted in the model updating the mechanical propertied of masonry with a correction factor equal to 2: f_m=4.00, τ_0 =0.035, E=1230, G=410 (N/mm^2). Stiffening of the floors was modelled accounting for diaphragm action on the relevant nodes.

Table 9 provides the results of modal analysis before and after interventions with respect to the fundamental mode of vibration in X, Y and Z direction. The strengthening interventions yielded an overall stiffening of the structure resulting in reduction of periods of vibrations.

	Before interventions		After interventions	
Mode	**T (sec)**	**Part. mass ratio (%)**	**T (sec)**	**Part. mass ratio (%)**
X	0.310	70.1	0.221	50.2
Y	0.503	86.3	0.364	86.9
Z	0.085	14.9	0.063	16.6

Table 9. Modal analysis results before and after interventions

The Seismic Risk Indexes in terms of PGA arising from Pushover analysis performed before and after interventions are given in Table 10. The proposed strengthening measures increased the seismic capacity of the structure yielding higher values of the indexes. In particular, the analyses performed without the vertical seismic effects show an increment of the index equal to +0.182 (+26%) while the analyses performed considering the vertical seismic effects show a lower increment of the index +0.080 (+13%). Therefore, the analysis carried out ignoring the vertical component of the seismic action tends to overestimate the benefic effects of the strengthening measures.

	Before interventions	**After interventions**	
Without Ez	0.700	0.882	+0.182 (+26%)
With Ez	0.609	0.689	+0.080 (+13%)

Table 10. Pushover analysis results. Seismic risk indexes ζ_E before and after interventions

According to current Italian regulations [6] the interventions of seismic improvement on existing buildings are accepted if they yield an increment of the Seismic Risk Index equal to 0.100. In this case the proposed strengthening measures would be acceptable if the vertical seismic effects are ignored, but they would not be enough if we consider those effects.

7 CONCLUSIONS

This study highlighted the crucial role played by the vertical component of the seismic action in the damages caused by recent earthquakes.

A methodology for considering the effects of the vertical seismic component in the seismic assessment of existing masonry building was introduced. The methodology in agreement with the current Standards allows to consider the vertical seismic effects in the local and global behavior of the structure. The local behavior governed by the out-of-plane response of walls is assessed through kinematic analysis taking into account the inertial forces induced by the vertical seismic component. The global behavior governed by the in-plane response of walls is assessed through pushover analysis, where the effects of the vertical seismic component result in a field of vertical inertial forces calculated through modal response spectrum analysis. The internal actions induced by the vertical forces are combined with those of static and incremental horizontal loading and considered in the safety verification of the elements. This leads to a global loss of resistance and displacement capacity of the structure.

The pushover analysis methodology was applied to the case study of a two-story masonry building considering different levels of masonry quality and several levels of seismic intensity. The analyses highlighted the negative effects of the vertical seismic component even for low values of the seismic action. The effects become more important as the ground acceleration increases, while the improvement of masonry quality can contrast and sometimes compensate the effects.

Further aspects of the methodology were introduced in order to make it viable for professional application in the assessment of real masonry buildings: (a) effects of compression and decompression cycles due to vertical seismic excitation and (b) capacity of the structure in terms of PGA considering the variability of the vertical seismic effects. The complete methodology was applied to the seismic assessment of a real masonry building evaluating the effects of possible strengthening interventions. The study confirmed the importance of considering the vertical seismic effects since the analysis carried out ignoring the vertical seismic component tends to overestimate the benefic effects of the proposed strengthening measures.

REFERENCES

[1] M. Mariani, Terremoto e Caos: un nuovo percorso di analisi del comportamento dei sismi, Imready, 2017 (in Italian) https://www.ingenio-web.it/18584-terremoto-e-caos-un-nuovo-percorso-di-analisi-del-comportamento-dei-sismi

[2] A. J. Papazoglou, A. S. Elnashai, Analytical and field evidence of the damaging effect of vertical earthquake ground motion, *Earthquake Engineering and Structural Dynamics*, **25**, 1109-1137, 1996

[3] Yin Jiang, Yi Weijian, The modal pushover analysis with the consideration of the vertical seismic effects. *Industrial Construction*, **39(5)**, 39-46, 2009

[4] M. Mariani, F. Pugi, Effetti negativi del sisma verticale sul comportamento delle pareti esistenti in muratura, Imready, 2017 (in Italian) https://www.ingenio-web.it/sfogliabile/EffettiSismaVerticale/Index.html

[5] M. Mariani, F. Pugi, A. Francioso. Sisma verticale: amplificazione della vulnerabilità degli edifici esistenti in muratura, Imready, 2018 (in Italian) https://www.ingenio-web.it/21336-sisma-verticale-amplificazione-della-vulnerabilita-degli-edifici-esistenti-in-muratura

[6] Ministero delle Infrastrutture e dei Trasporti. Decreto 17 gennaio 2018. Aggiornamento delle norme tecniche per le costruzioni, Gazzetta Ufficiale della Repubblica Italiana n. 42 del 20 febbraio 2018 (in Italian)

[7] Ministero delle Infrastrutture e dei Trasporti. Circolare. n. 7 del 21 gennaio 2019. Istruzioni per l'applicazione dell'Aggiornamento delle norme tecniche per le costruzioni, Gazzetta Ufficiale della Repubblica Italiana n. 35 del 11 febbraio 2019 (in Italian)

[8] Eurocode 8. Design of structures for earthquake resistance - Part 1: General rules, seismic action and rules for buildings, EN 1998-1, 2005

[9] Aedes Software, Aedes.PCM: Progettazione di Costruzioni in Muratura, Version 2019, www.aedes.it