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SEISMIC VULNERABILITY ASSESSMENT OF AN OLD STONE MASONRY BUILDING AGGREGATE IN SAN PIO DELLE CAMERE, ITALY

Rui MAIO[1]; Romeu VICENTE[2]; Antonio FORMISANO[3] and Humberto VARUM[4]

ABSTRACT

Stone masonry buildings, whose behaviour is known to be non-linear, is still one of the ancient and most common building typology in European historical city centres and medieval villages. Nowadays, in these historical centres, it is very difficult to analyse a building as an independent structure when, for example, neighbouring buildings share the same mid-walls. It is in this context that the necessity of studying building aggregates emerged. Moreover, the majority of these old buildings in old city centres have been designed without seismic concerns, and once overpopulated, are nowadays in need of structural rehabilitation taking into account their architectural acknowledged value.

This paper approaches the seismic vulnerability assessment of a stone masonry building aggregate located in San Pio delle Camere, which was slightly affected by the 2009 L'Aquila earthquake. The structure was assessed through static non-linear numerical analysis by using the 3muri® software. Moreover, simplified methodologies based on different vulnerability index formulations were applied to compare all these outputs obtained through distinct procedures.

Globally, the activities carried out have provided a clear framework on the seismic vulnerability of the examined building aggregate, supporting the vulnerability index based methodologies for a prompt and global assessment of buildings, which can be potentially useful for the development of large scale urban strategies and widespread retrofitting interventions.

INTRODUCTION

In Portugal, a great part of the existing building stock was not submitted to specific seismic design, especially after the 1st November of 1755 Lisbon earthquake, where the evaluation of structural safety conditions has been often forgotten or undervalued, perhaps due to the absence of significant seismic activity in recent decades (Ferreira et al., 2013). Moreover, ancient buildings are at this point in need of retrofitting. Additionally, in historical centres these buildings were weakened overtime due to economical interests and building adaptability, for example, with the openings on the façade walls and suppression of walls at the ground floor level, reducing the respective shear strength. Although being recognised as an important seismic zone in Europe, the absence of late important seismic events in Portugal, unlikely to the reality of other Mediterranean countries, has lead the Portuguese research community to learn and investigate from cross-border case studies. After the two notorious

[1] Researcher, Civil Engineering Dept. of the University of Aveiro, Aveiro, Portugal, ruiamaio@ua.pt
[2] Assistant Professor at the Civil Engineering Dept. of the University of Aveiro, Aveiro, Portugal, romvic@ua.pt
[3] Assistant Professor at the Dept. of Structures for Engineering and Architecture of the University of Naples "Federico II", Naples, Italy, antoform@unina.it
[4] Associate Professor at the Civil Engineering Dept. of the University of Aveiro, Aveiro, Portugal, hvarum@ua.pt

earthquakes which recently took place in Italy and highlighted the seismic vulnerability of old masonry buildings, Abruzzo 2009 and Emilia 2012, numerous Italian research projects based on post-earthquake observation data were developed, boosting greater concerns in countries with similar seismic risk.

According to the foregoing and in order to understand the structural behaviour of buildings in aggregates, a case study of a row building aggregate located in San Pio delle Camere (Abruzzo, Italy), affected by the 2009 April 6th earthquake occurred in L'Aquila and its districts, is herein presented and discussed (see *Figure 1*).

(a) (b) (c)

Figure 1. South (a) and north (b) and (c) façade of the modelled building aggregate located in San Pio delle Camere, Italy (Scheda di Aggregato, 2010)

Analysing this structure, whose typology is considered quite similar to the Portuguese stone masonry building typology, it was intended to obtain broader results for masonry buildings throughout southern Europe from structural typology similarity. Furthermore, this research sought a better knowledge of the behavioural differences between the seismic vulnerability assessment of individual buildings and building aggregates, as well as the reliability of more simplified methodologies applied for the evaluation of the seismic vulnerability of building aggregates. The building aggregate was modelled resourcing to 3muri® software developed by the STA DATA (2007) for seismic analysis of masonry and mixed constructions. According to the renowned classification for the seismic vulnerability assessment of buildings developed by Corsanego & Petrini (1990), both hybrid and indirect techniques were used. On one hand, static non-linear numerical analyses have been performed to obtain the capacity curves together with the prediction of the damage distribution caused by the input seismic action (hybrid technique). On the other hand, indirect techniques, based on different vulnerability index formulations, were used for a quicker assessment of the investigated case study propensity to damage under the earthquake action.

SAN PIO DELLE CAMERE

San Pio delle Camere is a small medieval village born in 1001 in the region of Abruzzo, about 25 km south from L'Aquila, elevated 800 meters above the sea level. The historical centre of this village was built along its main axis, Via del Protettore, where the surrounding buildings along are generally narrow, featuring low ceilings and often showing in-height irregularities. Recently included in the European Cultural Heritage Protection Program, San Pio delle Camere's historical centre is located in the upper part of the village, as the majority of the medieval Italian villages. During the last Century, several intrusive interventions were carried out, mischaracterizing both the architecture and morphology of the village.

The generalised characteristic of historical centres layout is the structural continuity of buildings (Carocci, 2001), frequently ensured when adjacent buildings, structurally connected to each other, are capable to induce either constraint vertical loads or horizontal thrusts among themselves (Binda et al., 2010). These blocks generally have the configuration and size defined by the urban layout of the village. Typically, in rural areas, as the historical centre of San Pio delle Camere, stone masonry buildings are smaller in their overall size, but they also have a smaller percentage between

the volume of openings and the overall volume of the building, being the quality of construction and of materials lower than the one found in urban stone masonry buildings.

In order to carry out the study of the seismic vulnerability assessment of any type of buildings, engineers need a certain survey accuracy level. In this sense, the University of Pisa has developed a detailed survey of building aggregates in this village (Mannari et al. 2011), resumed in the corresponding Scheda di Aggregato report, which contains the main input information needed for further seismic vulnerability assessment and evaluation, such as the aggregate typology and dimensions, conservation state, damage survey, plants and explanatory pictures.

SEISMIC VULNERABILITY ASSESSMENT OF THE BUILDING AGGREGATE CASE STUDY

The building aggregate, composed of six individual structural units (*S.U.*) from *A* to *F*, corresponding to six different dwellings, is herein assessed (see *Figure 2*). The urban delimitation of this row building aggregate, as most of the common typologies within Italian historical centres, is given by streets layout, in this case represented by Via del Protettore at north and one secondary street at south. From the west side, the building compound is delimited by a flight of stairs providing the discontinuity between the structural unit *S.U. F* and the adjacent building. The building aggregate has a total plane area of 319.5 m^2 and a total volume of 4535 m^3. The wall thickness of structural units varies approximately between 0.50 and 1.10 meters, while storey's height ranges between 2.60 and 3.20 meters. The north façade has two underground storeys, representing approximately 15.5% of the total volume of the aggregate, accounting higher stiffness when compared with the opposite façade. Moreover, openings, representing approximately 7% of the total vertical surface (façade walls), are not distributed evenly and their sizes vary widely.

(a) (b)

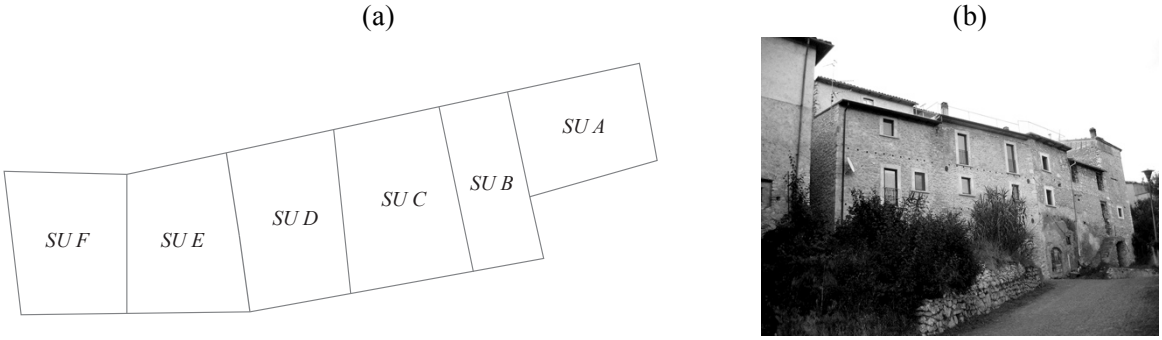

Figure 2. Structural units *S.U.* of the evaluated building aggregate (a) and the corresponding south façade (b)

With respect to structural materials, this building aggregate is mainly made of stone masonry, but some parts were built in other materials, which together with its intrinsic geometry, quantity and distribution of both openings and floors, raised some structural complexity. In fact, while structural units *S.U. C*, *S.U. D*, *S.U. E* and *S.U. F* are mainly in stone masonry, the structural units *S.U. A* and *S.U. B* have different materials varying in height such as stone masonry, reinforced stone masonry, cement block masonry and concrete (see *Table 1* and *Figure 3*). Due to the inexistence of *in-situ* experimental testing on stone masonry, in this work it was used the lowest value of Young's modulus, *E*, given by the NTC for stone masonry type *A*, since the stone masonry present in the aggregate under analysis was considered poor and irregular.

Table 1. Mechanical properties of structural vertical bearing structures

Materials	E [N/mm^2]	G [N/mm^2]	γ [KN/m^3]	f_m [N/cm^2]	τ [N/cm^2]	f_k [N/cm^2]	γ_m [-]
Irregular and rubble stone disorganised masonry	690	230	20	100.0	2.0	210.0	3.0
Cement block masonry	1400	350	12	111.1	7.0	77.8	3.0
Concrete C16/20	24167	11920	25	24.0	10.0	16.0	1.5
Reinforced masonry with irregular and disorganised stone	2175	725	20	185.2	3.7	129.6	3.0

(a) (b) (c) (d)

Figure 3. Vertical structure materials observed during the inspection phase. (a) Irregular fabric of stone masonry, (b) light cement blocks, (c) reinforced concrete and (d) reinforced stone masonry

The seismic sequence that struck L'Aquila province is well known all over the world and has been studied since back then by several experienced researchers. Further information regarding the 2009 L'Aquila earthquake can be consulted in the work developed by Salamon et al. (2010). The damage verified in San Pio delle Camere was not as severe as in L'Aquila and other surrounding villages due to the soil quality (rocky soil), which reduced the amplification of seismic waves. The damage intensity on this village was evaluated as I_{MCS}=V-VI (MCS scale).

The input seismic action was represented by an elastic horizontal response spectrum, defined according the required spectral parameters a_g, F_0 and T_c, attached in Table 1 of Annex B of the Italian seismic code NTC (2008) and a function of the site geographic coordinates of the aggregate building. These three parameter values (a_g=0.26g, F_0=2.37 and T_c=0.35s) are defined for a return period T_R equal to 475 years, considering the Life Safety limit state, ULS. To complete the response spectrum two additional factors were needed, the subsoil factor S_S and the topographic category S_T, calculated as suggested in the NTC. The elastic response spectrum was converted in the ADRS format (Chopra & Goel, 1999), representing the spectral acceleration S_a versus the spectral displacement S_d, for evaluating the seismic behaviour of the case study according to the Italian code NTC formulation.

HYBRID TECHNIQUE

The seismic vulnerability of the building aggregate was investigated through a hybrid technique based on the use of the Capacity Spectrum Method, CSM, to evaluate the performance point of the structure. Thus, in order to estimate fragility curves and damage discrete distribution, a formulation correlating the well-known *EMS-98* damage states with the bilinear capacity curve (expressing the yielding and ultimate capacity of the structure) obtained through non-linear numerical analysis, henceforth-named DLSF (Damage Limit State Formulation) was used.

The program 3muri® was the selected tool to perform pushover analysis on the stone masonry building aggregate, enabling the assessment of its global seismic response. This software, developed by the University of Genoa under the leadership of Sergio Lagomarsino, is

nowadays one of the most widespread software within its category for the seismic vulnerability evaluation in non-linear static field of masonry structures.

Using the Frame by the Macro-Elements (FME) method, in which macro-elements dimensions are a function of the global geometry of the aggregate, the dimension of the storeys, openings and the distances between openings. The formulation of masonry macro-elements emerged by observing the post-event effects in structures. 3muri® considers that structures can be efficiently represented as a combination of masonry panels constituted by spandrel beams and piers, subsequently represented by macro-elements with non-linear behaviour, connected by rigid nodes. This formulation reproduces the three principal in-plan collapse modes of a masonry panel, the bending-rocking, shear-sliding and diagonal shear cracking, with a limited number of degrees of freedom, allowing representing the seismic response of complex masonry structures with a very limited computational demand.

Although in Portugal this software is not widely known, there are some researches at the University of Aveiro and University of Minho, which have used this software to perform the mentioned analyses (Vicente, 2008; Marques et al., 2012; Ademović & Oliveira, 2012). *Figure 4* shows the resulting three-dimensional numerical view of the building aggregate and the East and West side elevations of the building aggregate, evidencing the slope between both longitudinal façades.

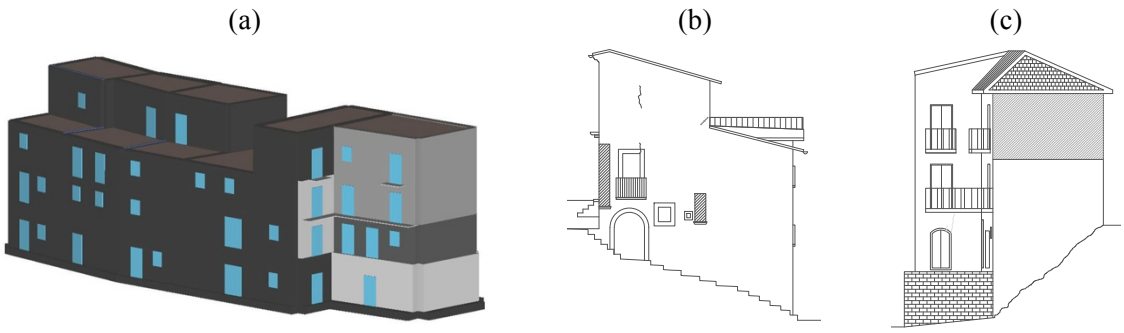

Figure 4. South façade of the building aggregate modelled with 3muri® (a), East (b) and West (c) side elevations of the building aggregate

Due to the structural complexity reflected by irregularity and heterogeneity of the model under analysis, different trial and error combinations to understand the influence of some parameters over the global behaviour of the building aggregate were performed. The activity purpose was to individuate the most unfavourable combination to be chosen. Generally speaking, this analysis includes a mass weighted displacement at the third floor controlled by barycentre node, N_{70}, accounts for lateral constraints effect on the north façade inferior walls (see Figure 4) and considers default fixed wall-floor constraints. From all the twenty-four different analyses performed using 3muri®, the most unfavourable one was obtained considering the longitudinal and transversal negative directions for a modal load distribution with no accidental eccentricity, which will be referred hereinafter as *analysis 1*. One other combination, *analysis 2*, was performed in the U_x and U_y positive direction for the same load pattern, but accounting accidental eccentricity effects.

Figure 5 illustrates separately the pushover curves for the longitudinal U_x and transversal U_y directions. In the reported pictures were plotted the previously mentioned *analysis 1* and *analysis 2*, as well as U_{xMass} and U_{y1stM} ones, gathering the remaining analyses into uniform and modal load distribution types, respectively. From these results, it is possible to observe a base shear force F_b value in the transversal direction U_y lower than the longitudinal direction one. This fact is explained by the percentage of resistant elements, which is significantly lower along the U_y direction. Moreover, base shear force values, F_b, vary between 3000 and 6000 kN for the U_x direction. Instead, in the U_y direction, F_b varies between 2000 and 4000 kN, approximately. With respect to the control node ultimate displacements d_n, these values ranged between 0.4-1.6 cm and 0.4-1.0 cm for the U_x and U_y directions, respectively.

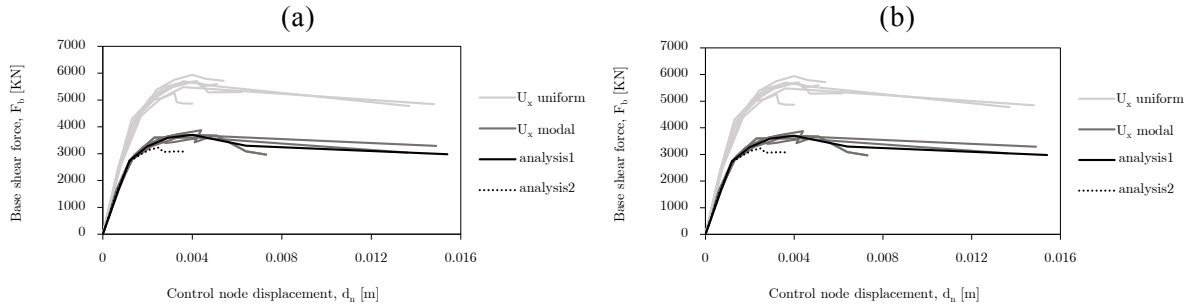

Figure 5. Pushover curves results for *analysis 1* (a) U_x and (b) U_y directions.

The observed base shear force values are quite unusual and extremely high for traditional stone masonry structures. A simple relation between the total base shear force and gravity loads has proven this extremely high value. In fact, this ratio changes between 26% and 60%, when for typical masonry buildings is estimated to be around 10%. Such value is likely to be justified by the small ratio between the volume of openings and the total volume of walls. Moreover, the large thickness of masonry walls, sometimes reaching a meter thick, and the short span between structural walls increased the stiffness and, subsequently, the base shear force, F_b. The modal load distribution was estimated roughly 2000 kN below the uniform one.

The main parameters achieved for the referred pushover analysis, shown in *Table 2*, resumes both yielding and ultimate capacity, ULS spectral displacements among other important output values, allowing the confrontation of the building aggregate response in both planar directions. It is relevant to underline that the performance point d_{max} values (also known as structural demand displacement), is used to estimate fragility curves. Thus, for *analysis 1*, d_{max} was evaluated as 0.33 cm for the longitudinal direction U_x and as 0.62 cm for the transversal direction U_y. Moreover, the displacement capacity of the structure d_u was equal to 0.50 cm and 1.54 cm for the U_x and U_y directions, respectively. The period T^* relative the equivalent SDoF system was evaluated as 0.12 s and 0.13 s for the U_x and U_y directions, respectively, being in accordance with the previous results, since lower periods means lower displacements values.

Table 2. Overall output values achieved through the application of the CSM method

		d_{max} [m]	d_u [m]	α_u [-]	Γ [-]	T^* [s]	F_y^* [kN]	d_y^* [m]	d_u^* [m]	A_y [g]	μ [-]
Analysis 1	$-U_x$	0.0033	0.0050	1.24	0.65	0.12	3891	0.0017	0.0076	0.39	4.53
	$-U_y$	0.0062	0.0154	1.62	0.86	0.13	3781	0.0016	0.0179	0.45	10.86
Analysis 2	$+U_x$	0.0065	0.0037	0.73	0.86	0.13	3590	0.0016	0.0043	0.36	2.69
	$+U_y$	0.0264	0.0046	0.26	1.08	0.26	1966	0.0026	0.0042	0.43	2.07

Fragility curves were developed considering the results from *analysis 1*, since they are likely more accurate in terms of spectral displacement and, subsequently, in terms of capacity curve. The formulation behind the fragility curves from **Error! Reference source not found.**, is explained in detail in the following bibliographic references (Vicente, 2008; Maio, 2013). From this figure it is possible to observe that damage grade D_{k2} in both directions prevails, being these conditional cumulative probabilities in the longitudinal direction U_x smoother than the U_y direction. In fact, this difference between both planar directions is associated to the value of d_{max}, which is larger in the transversal direction U_y, resulting the expectable damages probabilities slightly higher when compared to the longitudinal direction U_x.

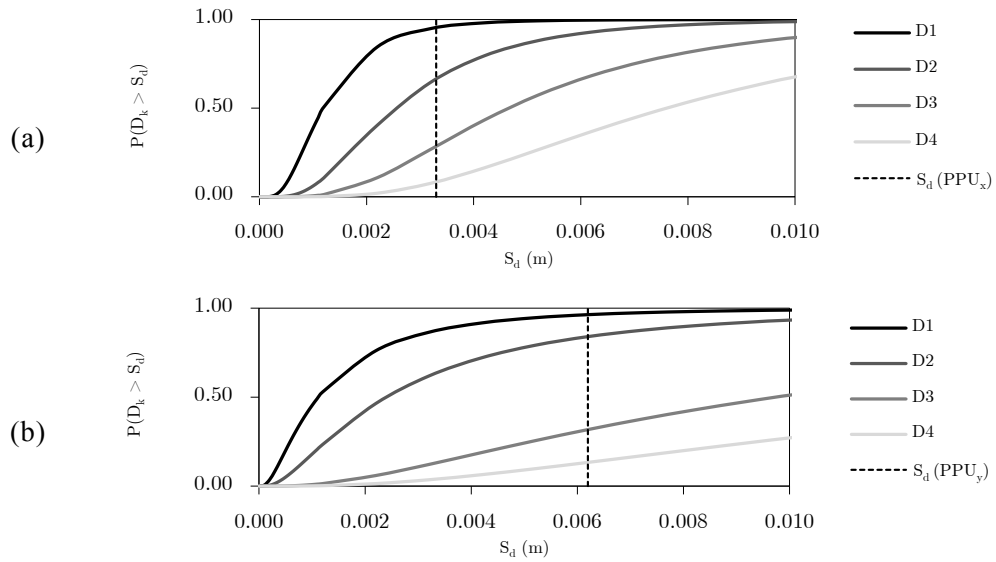

Figure 6. Fragility curves corresponding to the (a) U_x and (b) U_y directions

INDIRECT TECHNIQUE: INDIVIDUAL BUILDINGS ASSESSMENT

Within the indirect techniques two different proposals regarding the assessment of the seismic vulnerability of individual buildings were applied, both based on the original GNDT level II methodology (GNDT, 1994). Benedetti and Petrini (1984) developed the vulnerability function definition, which implied a deterministic correlation between the seismic action and the damage level. This binomial relationship was reached through extensive and detailed research and recent post-event observation of masonry buildings in the Italian territory. The characteristics that govern the seismic behaviour of masonry old buildings are treated as parameters, which must be evaluated in order to assess the vulnerability index value.

The methodology used for the individual vulnerability assessment of buildings was proposed by Vicente (2008) and has been used lately on large-scale assessment of some Portuguese historical centres such has Seixal (Ferreira et al., 2013). With this method the vulnerability index I^*_V, obtained by means of fourteen weighted mean structural parameters classified into four classes, from *A* to *D*, with independent class values C_{Vi} varying between 0 and 650. These parameters evaluates fourteen different aspects considered fundamental to describe the seismic behaviour, weighted by means of the p_i values, ranging from 0.5 up to 1.5 and representing the less or more importance in the building's vulnerability. I_V, varying from 0 to 100, results as the normalised value of the previous vulnerability index I^*_V. With this value, damage distributions were predicted to different seismic grades represented through the European Macroseismic Scale intensities $I_{EMS\text{-}98}$. As a further confrontation measure, a mean vulnerability index value I_{Vm} was calculated from the individual normalised vulnerability index I_V estimated for each structural unit.

Adopting the principles of a macroseismic methodology, it is possible to obtain the mean damage grade μ_D (see the following equations (1) and (**2**)) for different macroseismic *EMS-98* intensities (Grünthal, 1998). Once defined the vulnerability index *V*, varying between 0 and 1, and the ductility coefficient *Q* estimated for this masonry buildings (assumed equal to 2.0), which represents the ratio between the growth of damage and the seismic intensity, the mean damage grade curves can be obtained (Vicente, 2008).

$$V = 0.58 + 0.0064 I_V \tag{1}$$

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 6.25V - 13.1}{Q}\right)\right] \;;\quad 0 \le \mu_D \le 5 \tag{2}$$

Later on, Formisano's methodology for seismic vulnerability assessment of masonry building aggregates has been adopted (Formisano et al., 2010). They have also developed a vulnerability index methodology based on the same original procedure developed by Benedetti and Petrini, but accounting for fifteen parameters, wherein the last five parameters evaluate the influence of adjacent buildings upon the behaviour of each structural unit. Scores and weights of these additional parameters of the Benedetti and Petrini's form were evaluated through numerical analyses performed with the 3muri® software (Formisano et al., 2011). Similarly to Vicente's methodology, the vulnerability index I_V was estimated for each structural unit according to the evaluation of fifteen parameters, where the corresponding parameters are different from the previous methodology.

Moreover, for both methodologies, the normalised vulnerability index mean value I_{Vm} was calculated to be compared with the index values obtained for single structural units. Table 3 and *Figure 7* show the differences between the vulnerability index values obtained from the Vicente's method ($I_{Vvic,}$) and the Formisano's one ($I_{Vfor,}$). The mean vulnerability index values $I_{Vvic,m}$ and $I_{Vfor,m}$ were evaluated as 38.7 and 34.6, respectively by Vicente and Formisano methodologies.

Table 3. Structural units vulnerability index values for both used methodologies

	S.U. F	*S.U. E*	*S.U. D*	*S.U. C*	*S.U. B*	*S.U. A*
I_{Vvic}	60.4	39.0	39.0	33.3	23.8	36.5
I_{Vfor}	57.3	38.8	36.9	28.7	15.1	31.1

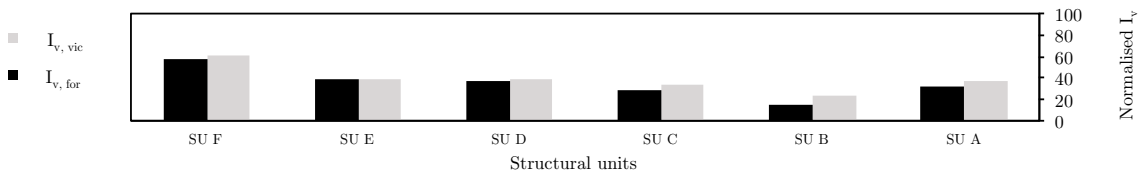

Figure 7. Comparison between Vicente and Formisano methodologies for each *S.U.*

INDIRECT TECHNIQUE: BUILDINGS IN AGGREGATE ASSESSMENT

The seismic vulnerability of building aggregates is becoming a widely recognised topic, since the influence of adjacent buildings is considered fundamental when interpreting post-event damages. Vicente (2008) and Formisano et al. (2011) developed the before mentioned methodologies based on the GNDT form (1994), but specifically developed for buildings in aggregate structural seismic assessment (Ferreira et al., 2012). The vulnerability index is calculated as the weighted sum of five parameters related to four classes C_{Vi} of growing vulnerability (from *A* to *D*). Each parameter evaluates one aspect regarding the seismic response of the building aggregate, assigning the vulnerability class through the analysis of different properties associated with mechanical, geometrical and inherent characteristics. Subsequently, for each one of these parameters a weight p_i is assigned, varying between 0.5 and 1.75, depending on the importance considered for each parameter. The value of the building aggregate vulnerability index I_{Va} (the following equation) ranges between 0 and 225. Finally, normalizing the previous value, by means of a weighted sum, the vulnerability index I_{Va} of the building aggregate is achieved.

The vulnerability index I_{Va} estimated for the building aggregate under analysis was 38.3. On one hand this value is very close to the Vicente's mean vulnerability index $I_{Vvic,\ m}$, while, on the other hand, the aggregate vulnerability index value I_{Va} differs approximately 10% when compared to Formisano's mean vulnerability index value $I_{Vfor,m}$. *Figure 8* illustrates the fragility curves representing the mean damage grade values μ_D for different seismic intensities obtained through the aggregate vulnerability index I_{Va}.

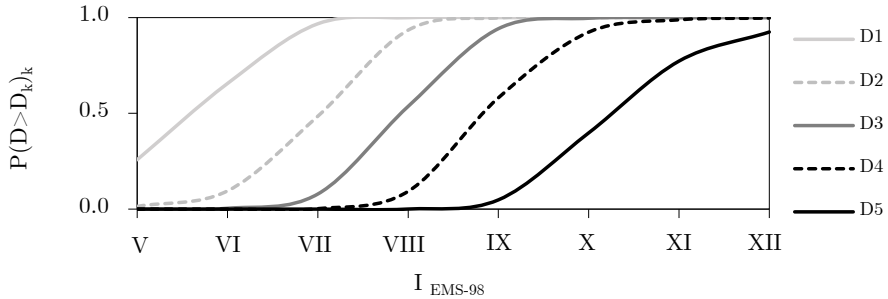

Figure 8. Fragility curves for different *EMS-98* macroseismic intensities, estimated for the aggregate vulnerability index value I_{Va}

COMPARISON BETWEEN METHODOLOGIES

The correlation between *EMS-98* scale and the Peak Ground Acceleration value, PGA, is defined by the following equation *(3*), developed by Lagomarsino and Giovinazzi, which allows for the comparison between the damage distribution of both hybrid and indirect techniques (Giovinazzi & Lagomarsino, 2006):

$$a_g = c_1 c_2^{(I-5)} \tag{3}$$

where a_g is the peak ground acceleration in g, I is the *EMS-98* macroseismic intensity value, c_1 is the coefficient which defines the PGA value for a default macroseismic intensity V and c_2 defines the slope of the correlation curve. Correlation laws developed by Guarenti-Petrini, Margottini and Murphy-O'Brien (Guarenti & Petrini, 1989; Margottini et al., 1992; Murphy & O'Brien, 1977) were used to estimate the corresponding EMS-98 intensity I, which had led to equivalent EMS-98 macroseismic intensities from VIII to IX.

Figure 9 illustrates the damage distribution for *EMS-98* macroseismic intensities VIII and IX of all the applied methodologies together. Facing $I_{Vvic,\ m}$ and $I_{Vfor,\ m}$ damage distributions, it was observed a maximum deviation in D_{k3} equal to 7.3% and 7.4%, for $I_{EMS\text{-}98}$ = VIII and $I_{EMS\text{-}98}$ = IX, respectively. When comparing $I_{Vvic,\ m}$ with I_{Va} these difference was substantially reduced to 1.1% for $I_{EMS\text{-}98}$ = VIII. Instead, when comparing $I_{Vvic,\ m}$ to DLSF distributions, differences resulted more noticeable, since they have reached 25.1% and 51.8% for the U_x direction and 26.8% and 48.8% for the transversal direction U_y. When compared to $I_{Vvic,\ m}$ general results, smaller deviations were obtained for most of the comparisons with $I_{Vfor,\ m}$. For $I_{EMS\text{-}98}$ = VIII the maximum deviation value of 6.4% was found comparing it to I_{Va}. The maximum difference for $I_{EMS\text{-}98}$ = IX was remarkable, since it goes against the observed trend, in which higher deviation values were expected for this intensity, being in this case 0.4%. The comparison between $I_{Vfor,\ m}$ and the longitudinal direction of DLSF reached the largest difference of 18.5% and 45.6% for $I_{EMS\text{-}98}$ = VIII, whereas for $I_{EMS\text{-}98}$ = IX the maximum difference was 19.5% and 46.4%, respectively. Finally, comparing I_{Va} for both directions of DLSF distribution, is was registered a maximum deviation of 24.2% and 46.0% in the U_x direction, while in the U_y one these differences were 25.9% and 46.6%, respectively.

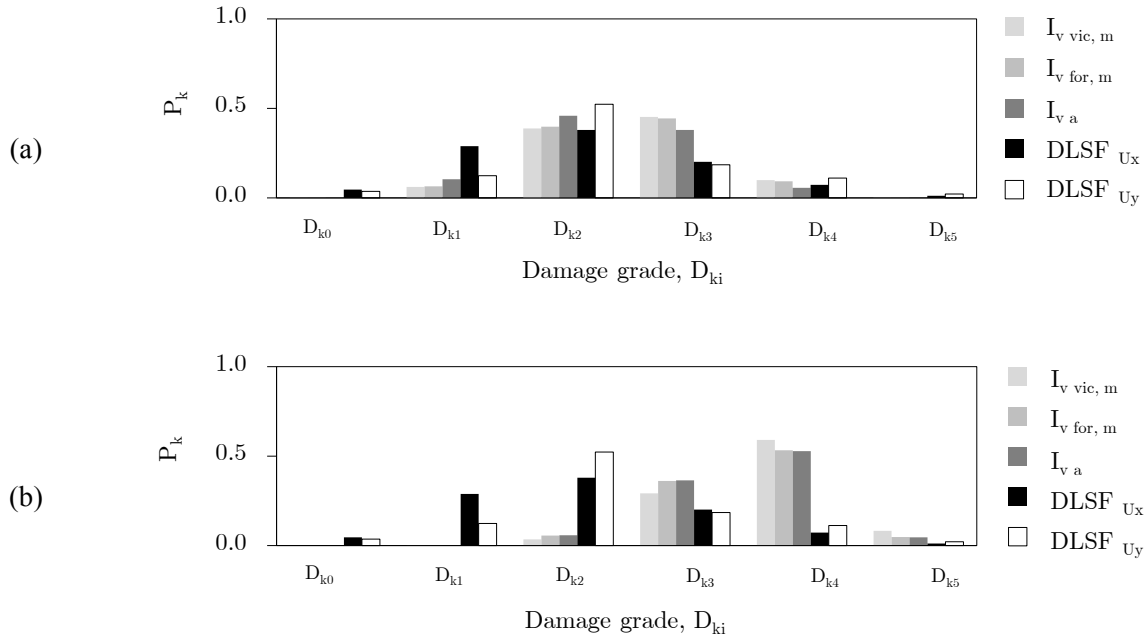

Figure 9. Damage distribution comparison between the three analysed vulnerability index formulations for (a) $I_{EMS\text{-}98}$ = VIII and (b) $I_{EMS\text{-}98}$ = IX macroseismic intensities

FINAL COMMENTS

Building aggregates result as a middle term scale class of buildings, whose optimal assessment should embrace numerical analysis for a more detailed investigation, depending on the objective of the project in hands. The accuracy in the outcomes obtained through non-linear static analysis was found clearly influenced by the input analysis parameters, such as the mechanical and geometrical properties of the structure. Therefore, the structural survey accuracy is reflected in the analyses results.

With respect to the numerical analysis, while in-plan irregularities were successfully overcome, causing no numerical issues, in-height irregularities were considered more problematic, since they affect the nodal displacements, which can somehow compromise the safety verification and the analysis convergence. The results obtained indicate the transversal direction as the most vulnerable, which is coherent with the real damage verified in the *in-situ* inspection, in which transversal walls were significantly more damaged than longitudinal ones. The lower node displacement values of the pushover analysis were a consequence of the effort of representing the behaviour of the slope between the two principal façades through modifying the lateral supporting condition of the structure. This assumption was clearly found too conservative and it might be related with some inconsistencies regarding pushover and capacity curves. As a general comment, it is sensible to affirm that the damage distribution predicted by the pushover curves are somehow assimilated to the real damage distribution occurred in the aggregate, both in terms of extension and type of failures.

Indirect techniques were carried out to compare their accuracy when compared with hybrid techniques. With respect to the individual structural assessment, the methodology proposed by Vicente shown slight differences by excess in structural units vulnerability index, when compared to Formisano et al. (2011) methodology. For structural units *S.U. F*, *S.U. E* and *S.U. D* the highest vulnerability index values of 60.4, 39.0 and 39.0 for Vicente's methodology were attributed, while for Formisano's methodology these values were 57.3, 38.8 and 36.9. So, for structural units *S.U. D*, *S.U. E* and *SU F* a great resemblance between results has been shown, with deviations below 5.5%. On the contrary, it was found important deviations between these two methodologies for structural units *S.U. A*, *S.U. B* and *S.U. C*, which identifies the parameters

relative to masonry material heterogeneity as the most different between both methodologies. Moreover, with respect to row end buildings *S.U. A* and *S.U. F* higher vulnerability index values are associated, in agreement with the later studies conclusions regarding building aggregates vulnerability assessment. Mean vulnerability index values were estimated for the previous methodologies in order to construct a prediction of the building aggregate vulnerability suitable to be compared with the building aggregate vulnerability index formulation proposed by Vicente and Formisano. Therefore, on one hand, the vulnerability index methodology designed for building aggregates approximates very accurately the mean vulnerability index values of Vicente's methodology. On the other hand, the vulnerability index of Vicente and Formisano were found to be in conformity for comparatively regular buildings.

The conversion of the mechanistic approach using the *EMS-98* macroseismic scale was found to be a reasonable way of establishing comparisons with indirect approaches. Thus, it was possible to conclude that for a seismic intensity I_{EMS-98} = VIII, indirect techniques were found more representative of the real damage distribution in the building aggregate, from which aggregate vulnerability index methodology has shown as the less conservative of the three vulnerability index methodologies. With respect to the I_{EMS-98} = IX, indirect techniques revealed to be too conservative, while, on the contrary, mechanic techniques failed the approximation by default. These comparisons between hybrid and indirect techniques were found to be somehow inaccurate for extreme macroseismic intensities and very similar for medium macroseismic intensities. It was evident that data accuracy has subsequent implications on numerical analysis outputs, which can lead to unreliable results and interpretations. These computational analyses should be compared to quick vulnerability assessment methods in order to detect possible problems of numerical model environment. To avoid this, scientists should be aware and conscious if the knowledge level and survey related to a generic study gathers all data necessary to obtain feasible results. When this knowledge requirement is considered insufficient, it is preferable to conduct the analysis through empirical methodologies, which are proven to give satisfactory predictions about both damage predictions and seismic vulnerability assessment of either individual buildings or building aggregates.

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