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ORIGINAL PAPER

Local- and global-scale seismic analyses of historical masonry compounds in San Pio delle Camere (L'Aquila, Italy)

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Abstract Masonry building aggregates are large parts of the Italian building heritage often designed without respecting seismic criteria. The current seismic Italian code does not foresee a clear calculation method to predict their static nonlinear behaviour. For this reason, in this paper firstly, a simple methodology to forecast the seismic response of masonry aggregates in *San Pio delle Camere* (L'Aquila, Italy) has been set up starting from the provisions of the Italian Guidelines on Cultural Heritage. The implemented procedure has been calibrated on the results of two FEM structural analysis programs used to investigate three masonry building compounds. As a result, a design chart used to correctly predict the base shear of aggregate masonry units starting from code provisions has been set up. Later on, the large-scale seismic vulnerability and damage appraisal of the inspected historical centre has been done on the basis of a quick methodology, already implemented and experienced by the author in some historical centres of the Campania region. The analysis result was a numerical correlation between vulnerability index and mean damage grade of examined building compounds. In particular, a damage forecast under numerical way has been firstly estimated and then compared with the real one. The post-earthquake scenario has represented an ideal term of comparison for effectively testing the reliability of the employed technique, which should be further extended to other Italian historical centres.

Keywords Masonry building compounds · Static nonlinear analysis · Italian Guidelines on Cultural Heritage · Design chart · Damage curves · Large-scale vulnerability and damage evaluation

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1 Introduction

The historic centre built-up has always been not only a response to housing need over time, but also the testimony of centuries of civilization and culture, now judged as a touristic and economic irreplaceable resource. Masonry buildings represent a large part of the Italian building heritage, designed to withstand vertical loads and any horizontal forces induced by vaults or arches without respecting seismic criteria. So, for the analysis of these structures, there is almost always the trend to examine their seismic behaviour on the basis of unclear criteria. In particular, the case of masonry building aggregates represents the norm within roughly all Italian historical centres (Benedetti and Petrini 1984; Angeletti et al. 1988; Guagenti and Petrini 1989; Casolo et al. 1993; Giuffre 1993; Dolce et al. 2004; Asteris et al. 2016).

Aggregated buildings represent, in fact, an important and typical peculiarity in many Italian old town centres. Most common aggregated building type are continuous curtains of masonry buildings developed along streets with different total height, storey height, number of floor, erection age and structural typology. Generally, aggregated buildings can show a complex vertical and/or horizontal development, so giving rise to building groups with different heights and shapes. Reasons of this variability came by the spontaneous erection way, without rules, to build constructions during different historical ages.

Analysis of historical aggregated buildings represents an important and very innovative issue to be inspected after recent seismic events affecting the Italian region. L'Aquila earthquake and, recently, the Emilia-Romagna one demonstrated that aggregated buildings generally show a group behaviour which improves seismic performances of the constituent structural units, also when they are made of low-quality masonry (Formisano 2012a; Formisano et al. 2010a, b, 2011, 2015; Indirli et al. 2013).

According to the recent relevant codes on building aggregates, such as the Italian O.P.C.M. 3431/05 (2005), M.D. 14/01/08 (2008) and M.C. 02/02/09 n. 617 (2009) standards, it is worth to be noticed that an aggregate is composed by a group of not homogeneous structural units interacting with each other during earthquakes. So, an aggregate is made by several buildings, which have a more or less efficient connection each to other. In fact, aggregated buildings can also be defined as "the combination of different units more or less connected among them that create (at least in apparent way) a unique entity difficult to be divided in parts with independent structural behaviour". For these reasons, the investigation purpose is not the entire aggregate only but also its parts, which are called "structural units" (S. U.), having a unitary and homogeneous behaviour towards static and dynamic loads.

In the literature, different approaches have been presented for studying the behaviour of structural units grouped into masonry building compounds (Binda and Saisi 2005; Carocci 2012; Da Porto et al. 2013; Dolce et al. 2006; Maio et al. 2015; Pagnini et al. 2011; Pujades et al. 2012; Ramos and Lourenço 2004; Senaldi et al. 2010).

Moreover, interesting and relevant standard provisions used for historical masonry buildings are the "Guidelines on Cultural Heritage" (MiBAC 2011). Such a standard, usually employed for isolated constructions, provides indications to both evaluate and reduce the seismic risk of protected cultural heritage according to the recent seismic Italian code (M. D. 2008). In particular, in order to evaluate seismic safety of mentioned buildings, three seismic analysis levels have been set up: (1) LV1, used to assess the seismic safety of protected heritage at large scale; (2) LV2, used for evaluating local interventions (first mode mechanisms) on building limited parts that Italian M.D. 08 defines as "reparation or local intervention" techniques; and (3) LV3, used either to design interventions influencing the whole structural behaviour (defined by M.D. 08 as "upgrading or retrofitting interventions") or to perform an accurate building seismic safety evaluation.

On the basis of these premises, the idea developed in the current paper is to extend the indications of the above Guidelines to the seismic behaviour appraisement of some historical building aggregates located in *San Pio delle Camere*, a little town in the district of L'Aquila (Italy).

In the current paper, both local detailed analyses on some case studies of historical aggregates and global simplified investigations on a large building stock of the mentioned town have been performed in the seismic vulnerability and damage fields.

First of all, in the framework of local analyses, a simple nonlinear methodology has been set up on the basis of calculation program results aiming at plotting simplified pushover curves of both the single structural units and the building compound. Moreover, damage curves of both isolated units and aggregated ones have been plotted in order to show the behavioural differences in the former when they are enclosed within building compounds.

On the other hand, as global-scale analyses, a quick methodology for large-scale seismic vulnerability and damage assessment has been applied to the examined Italian historical centre with the final purpose to evaluate its damage state after seismic events with different intensities. In addition, as a further result of the study, the proposed method has been also validated by the comparison between the forecast damages and those really occurred after the 2009 L'Aquila earthquake.

The final study target, which represents the research future development, is to apply the proposed analysis methods to other Italian historical building compounds aiming at both assessing the fragility curves of single units and aggregated (heading, corner and intermediate) ones and foreseeing their damages under earthquakes with different grades. As a result, the beneficial or detrimental effect deriving from grouping in aggregate will be shown for different types of structural units belonging to clustered buildings.

2 The *San Pio delle Camere* old town centre

San Pio delle Camere is a little town with mediaeval origin placed in the district of L'Aquila at the mountainside of the *Monte Gentile* along the Aterno Valley (Fig. 1). The adjectives "chambers" ("delle Camere" in Italian) or "caves" refer to the characteristic caves located under the constructions, constituting shelters for the flocks of nearby Peltuinum, and were introduced in 1600 to distinguish this village from others having the same name.

The old nucleus of the town, developed around the St. Pio church depicted in Fig. 2, was destroyed in 1424 by the troops of Braccio from Montone and rebuilt in the sixteenth century. In the same figure, a typical fortress of Abruzzo, called "Castles fence", which was built during the Renaissance Age, is visible on the top of the hill.

Nowadays, *San Pio delle Camere* consists of two parts: the historic nucleus and a more modern zone, the latter being composed of anonymous constructions.

On the other hand, the historic nucleus has a mediaeval tissue based on a process of an irregular urban growth, from the ancient times up to the present days, which does not allow to clearly distinguish the ancient pattern. Furthermore, the town built-up develops on slope soils, following the shape of the contour lines and the road layouts (Fig. 3). Thus, the aggregation of buildings in slope has characterized the typical constructions of *San Pio delle Camere*, the so-called profferlo houses, which are the town typical reference structures.

Fig. 1 Landscape of *San Pio delle Camere*

Fig. 2 The S. Pio Church in the historical centre of *San Pio delle Camere*

These mediaeval houses are generally made of two or three overlapping cells, which were connected by external masonry stairs, called "profferlo" in the Italian language. Usually, the ground floor was used to be a storage or a farm with independent entrance, whereas other floors were used for residential purpose. Generally, the number of overlapping cells depends on the ground natural slope (Fig. 4) (Ceradini 2003).

An interesting characteristic of the built-up is the recurrent presence of contrast arches among different building compounds, which were erected in the past to prevent the out-of-plane collapse mechanisms of masonry walls (Fig. 5). Moreover, in the old town centre, aggregated buildings on staggered levels are placed (Fig. 6). Masonry texture is not regular, but in spite of this, it shows a good apparatus with some distinctive features, like

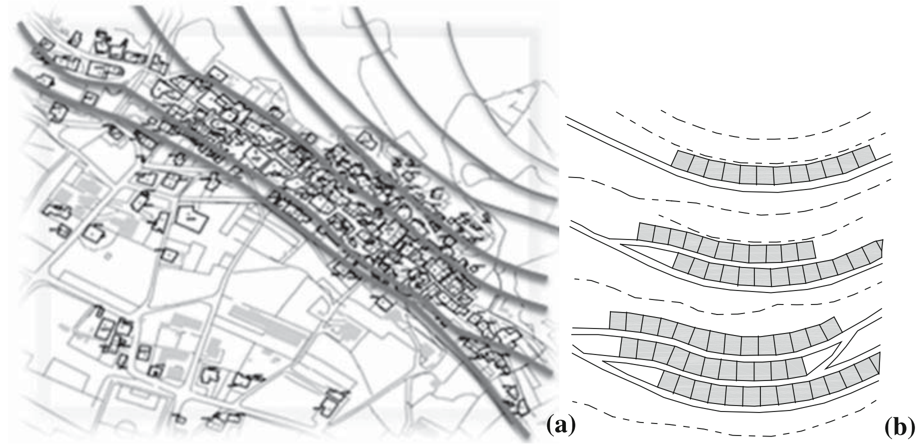


Fig. 3 The *San Pio delle Camere* building map (**a**) and the building aggregation parallel to the contour lines (**b**)

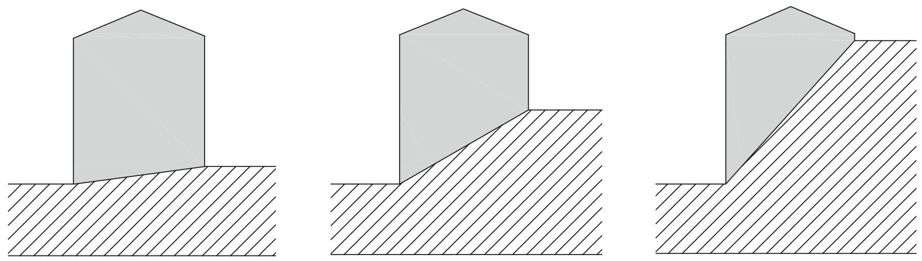


Fig. 4 Positions of buildings as respect to the ground natural slope (Ceradini 2003)

medium-size stones, horizontal layers and small dimension stones guarantying the contact among all elements and, therefore, the wall continuity (Formisano 2012b) (Fig. 7).

Buildings have three floors at most, whereas only in few cases, they developed on four levels. Low homogeneity with original building parts is observed in raised volumes that are realized with either full or perforated bricks or concrete blocks. Single-layer, cross, ribbed and barrel vaults, sometimes under a lowered configuration, are the most common horizontal structures. The most recurrent horizontal plane structures are timber floors in very deteriorated conditions. Other most recent floor kinds are those with either steel beams or reinforced concrete joists, both of them coupled with hollow brick tiles. One or two pitches wooden floors, in some cases showing thrusting behaviour, represent the main roofing structure.

3 Local-scale analyses

3.1 Foreword

Aggregated buildings situated in the *San Pio delle Camere* old town centre showed diffused structural damages due to the 6 April 2009 earthquake (Formisano et al. 2013).

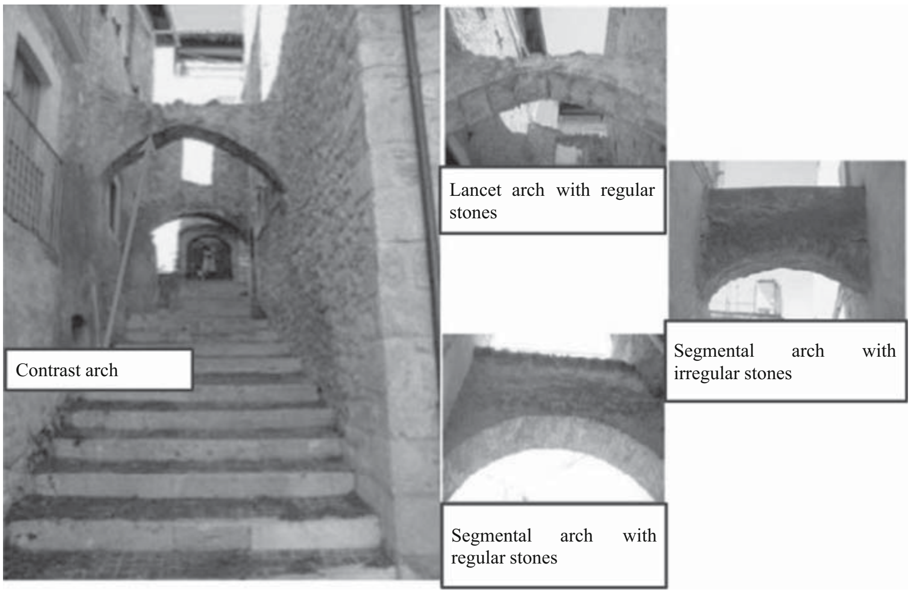


Fig. 5 Contrast arches in the historic centre

Fig. 6 Map of the historical building compounds

Three building aggregates with two (type A), four (type B) and six (type C) structural units have been selected as case studies in the investigation area (Formisano 2016).

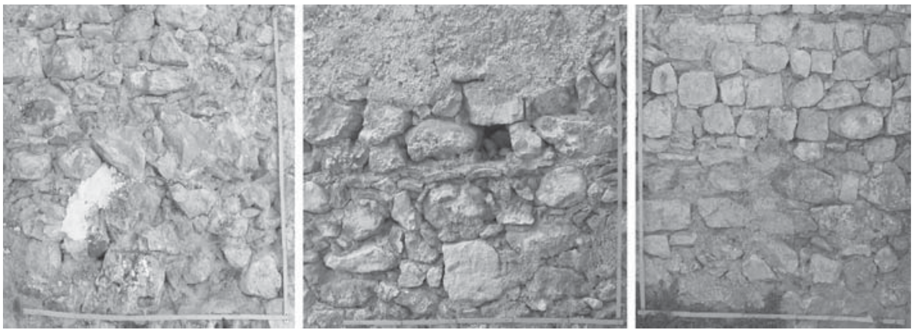


Fig. 7 Typical masonry textures

Aggregate type A is based on an 80% inclined ground. This justifies the presence in every units of a basement floor connected with underground caves. It is developed on a surface of 10.10 m × 5.20 m and has a height of 8.40 m over the ground on its south side. As illustrated in Fig. 8, different kinds of floors are placed in the building compound: vaults and timber floors at the first level and timber floors at the second level only. Roofing is represented by lightly thrusting timber structures.

Aggregate type B is also based on a very strong slope soil (Fig. 9) and can be inscribed into a 21.0 m × 8.0 m rectangle. It is formed by four structural units, three of them developing on three levels and only one (S.U. type C) on four floors. The aggregate geometrical configuration is shown in Fig. 10, where the plan layouts and an external view are plotted. Structural units are made of local masonry composed of irregular-shaped stones sustaining barrel and pavilion vaults. Building facades result to be aligned, and staggered floors are missing.

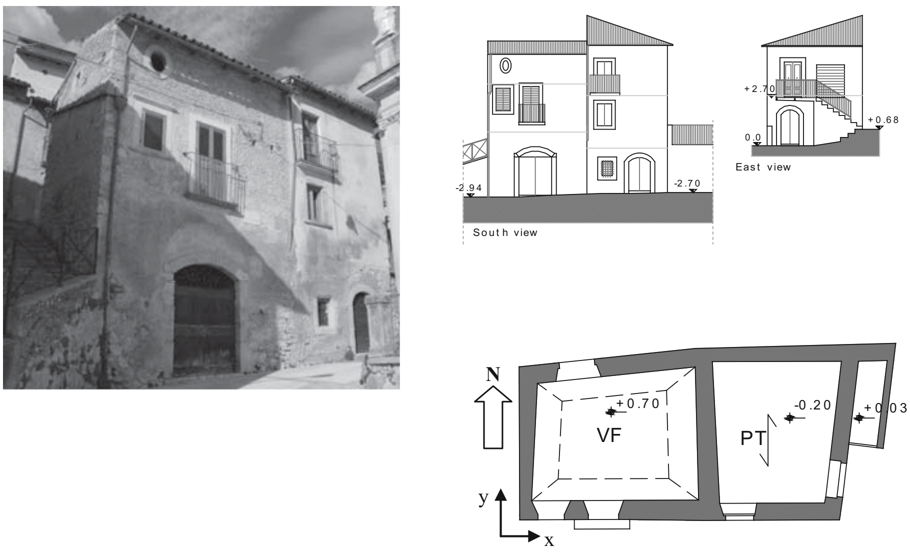


Fig. 8 Aggregate type A: main view and architectural drawings

Aggregate type C is based on a 65% inclined ground and can be inscribed into a 34.0 m × 14.0 m rectangular area. The aggregate structural units are made of a limestone masonry with drafted and irregular stones, having poor quality transverse tie and typically used in a lot of building aggregates situated in the L'Aquila neighbourhoods. Different floor types are located at various building levels, as illustrated in Fig. 11. All these floors, typically diffused in the examined area, have a good connection degree with loaded walls. Roofing are generally made of wooden pitched trusses, and some structural units were interested by renovation interventions based on metallic steel ties connecting parallel walls. In some specific cases, cracked stones have been substituted with new concrete bricks. Occasionally, original floors were replaced by reinforced concrete ones and timber roofing was sometimes substituted with reinforced concrete coverages.

Materials mechanical properties have been directly obtained from results of experimental tests conducted on a historical building of L'Aquila (Borri et al. 2012; Candela et al. 2012) (Fig. 12). Such tests have provided the following mechanical features: $\gamma_m = 19$ kN/m^3, $f_m = 210$ N/cm^2 (compression resistance mean value), $f_{vm0} = 4.55$ N/cm^2 (shear resistance mean value without axial force), $E = 856$ MPa (normal elastic modulus) and $G = 342$ MPa (tangential elastic modulus). From analysis of the above experimental data, it is apparent that the compression and shear strength values are greater than code ones, whereas Young and shear moduli are within the standard limits.

Design values of compression and shear strengths (f_d and f_{vd0}) have been obtained by penalizing the mean values achieved from experimental tests through both the partial safety factor γ_m (material coefficient) and the confidence factor CF (depending on the building knowledge level), as prescribed by the actual Italian standards.

In the case under study, a CF = 1.35, corresponding to a limited knowledge level LC1, has been assumed for existing buildings. This choice is justified since only the geometric survey is available for building aggregates under study, as well as both limited material in situ checks have been carried out and few architectural details have been investigated. On the other hand, Italian Circular no. 617/09 (2009) specifies that γ_m is one when non-linear static analyses are performed.

3.2 Numerical modelling and theoretical analysis

The study purpose is to implement a simplified procedure for seismic vulnerability assessment of historical masonry aggregates. This has been set up through the accurate numerical modelling and analysis of selected aggregated buildings, which has been carried out in two analysis phases.

Fig. 9 Main views of the aggregate type B

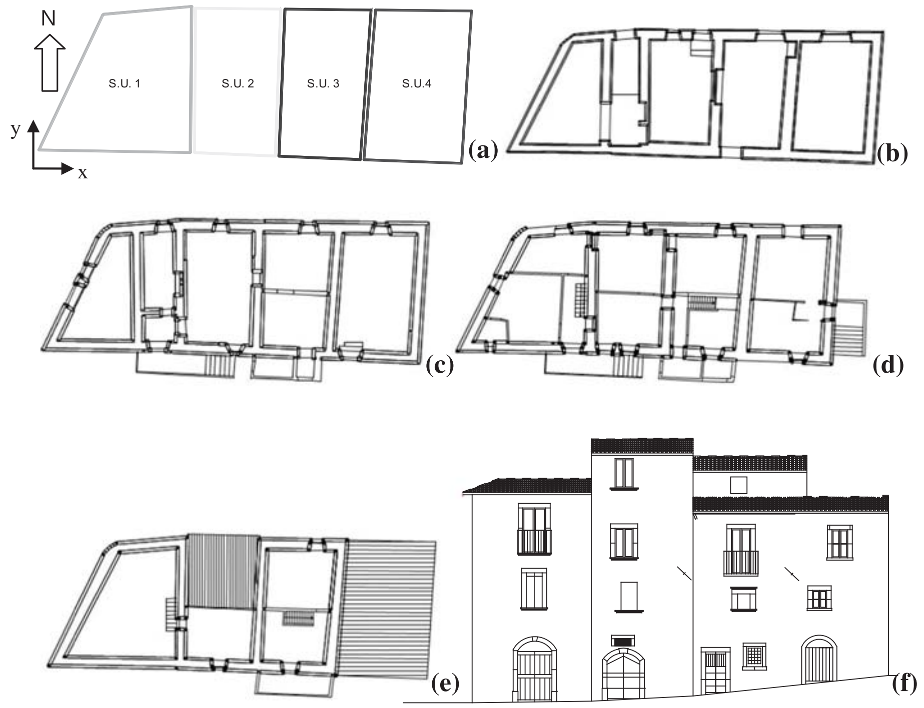


Fig. 10 Aggregate type B: general plan layout (**a**), underground floor (**b**), ground floor (**c**), first floor (**d**), second floor (**e**) and north side view (**f**)

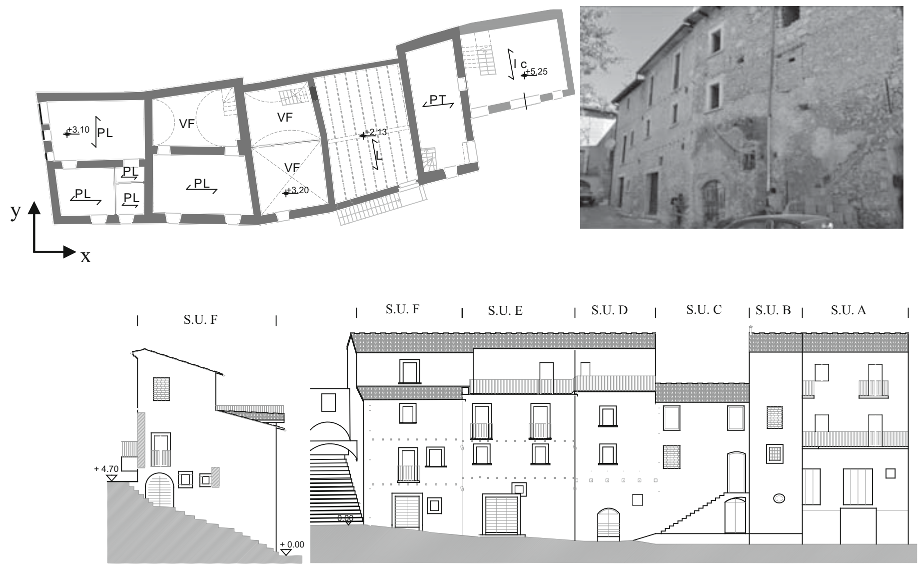


Fig. 11 Aggregate type C: main view and drawings

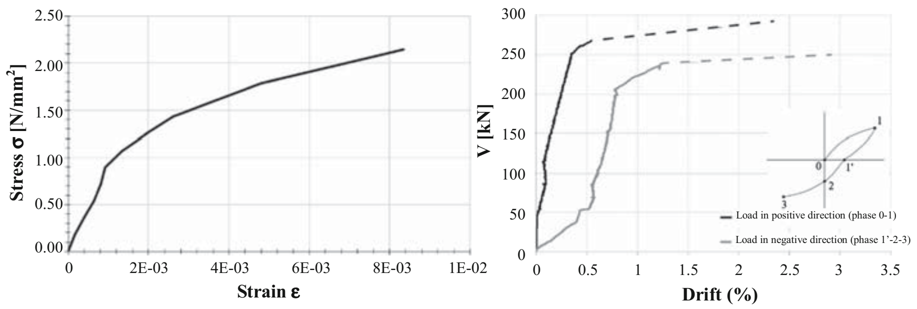


Fig. 12 Experimental responses of L'Aquila masonry panels (Borri et al. 2012; Candela et al. 2012)

In the first phase, the SAP2000 analysis program (CSI 2013) has been applied only to the aggregate type A in order to mainly assess the floor stiffness, which is difficult to be evaluated when flexible horizontal structures are of concern. Afterwards, in the second phase, the 3MURI program (S.T.A.DATA 2009) dedicated for seismic vulnerability assessment of masonry buildings has been used for examining all the study aggregates. The results achieved from numerical analyses have conducted towards a simple indication how to predict shear strength of examined structures starting from the basic resistance value achieved for historical buildings from the *Italian Guidelines for cultural heritage seismic hazard evaluation and reduction*.

Finally, damage curves of isolated units and units within aggregates have been directly derived from numerical analyses performed on the study aggregates. For the sake of example, such curves are herein presented for the aggregate type B.

3.2.1 The equivalent frame model

In the beginning phase, the equivalent frame model technique recommended by seismic Italian codes for reinforced concrete and steel framed structures has been used to model masonry buildings with beam elements through the SAP2000 software. Differently from what happens in RC framed structures, when frame modelling is used for masonry buildings, vertical beam elements representing masonry walls into two principal directions cannot be aligned (Fig. 13).

An important modelling aspect is related to the floor in-plane stiffness evaluation, which appears difficult to be assessed for deformable horizontal structure types, which are conceptually not amenable to modern rigid floors. In fact, due to both their configuration and the lack of a continuous armed slab, the deformable floors cannot be regarded as infinitely rigid in their plane. So, in the numerical model, shear stiffness of these floor types has been taken into account through their modelling with two diagonal trusses arranged according to a St. Andrew's cross-configuration.

In the current study, lumped plasticity modelling has been implemented for the equivalent frame members. In this way, the progressive greater deformability connected to the plastic behaviour extension is not considered, but the material nonlinearity related to the element plasticization is still taken into account.

Examined resistance domains are axial compression–bending moment for masonry piers and diagonal and sliding shear for masonry piers and spandrels. Plastic hinge diagrams are qualitatively depicted in Fig. 14, where the hinge rotation ultimate capacity is

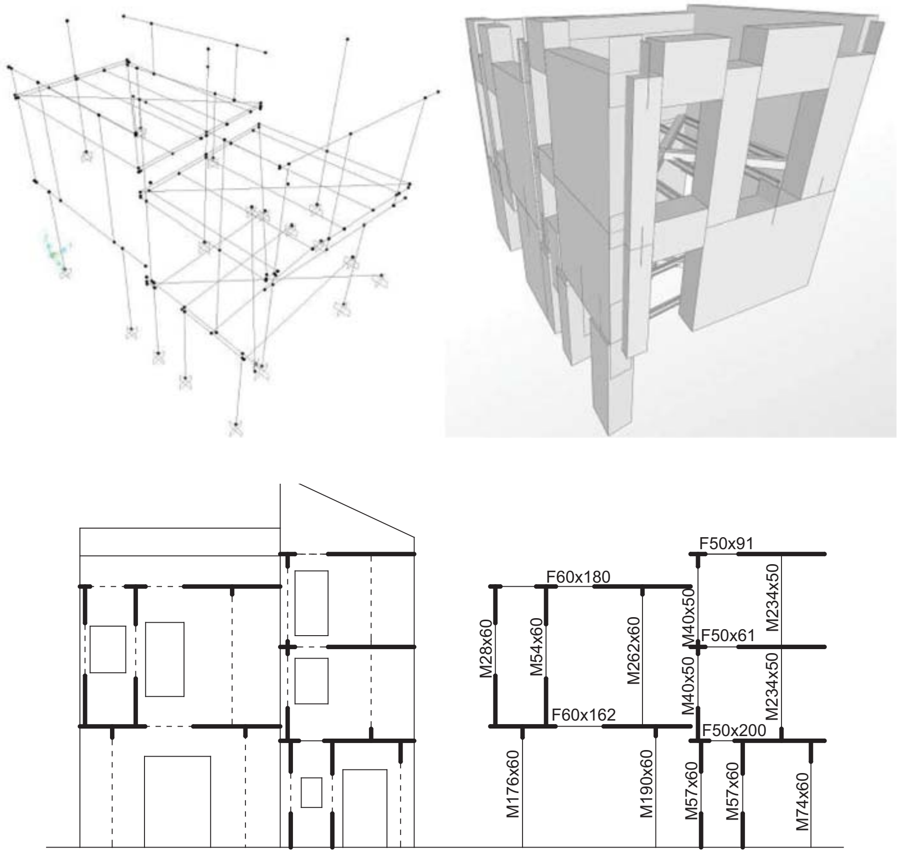
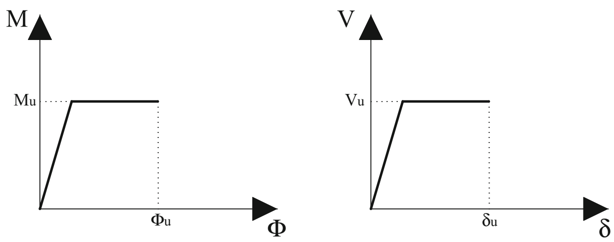


Fig. 13 Equivalent frame model of the aggregate type A

Fig. 14 Plastic hinge laws for masonry structural elements



defined in accordance with legislation indications on the basis of the following deformation limits:

- $\Phi_u = 0.008$ for masonry piers subjected to both compression and bending moment (0.006 for existing buildings);
- $\delta_u = 0.004$ for masonry piers and spandrels subjected to shear.

3.2.2 The macro-element model

The macro-element model has been implemented by using the 3MURI calculation software for modelling the three inspected aggregates. Starting from this modelling type, also in this case, a three-dimensional equivalent frame is used to model masonry walls, obtained by assembling all together deformable resistant elements (masonry piers and spandrels) with rigid nodes.

The program gives to the walls the role of resistant elements towards horizontal and vertical loads, whereas horizontal structures have the task both to distribute the vertical loads they receive and to share the horizontal actions to relevant masonry walls on the basis of their in-plane stiffness. About horizontal actions, the used modelling approach neglects the contribution of walls having own plane perpendicular to the load direction due to their considerable flexibility (Fig. 15).

3.2.3 Application of Italian Guidelines on Cultural Heritage

After definition of numerical models, the simplified LV1 method proposed in the Italian Guidelines on Cultural Heritage for "palaces, villas and other structures with intermediate bearing walls and horizontal elements" has been applied to the case studies under the hypothesis that structures have a box-like behaviour (MiBAC 2011).

The procedure assumes vertical load-bearing masonry walls in every direction and hypothesizes that the collapse occurs when the average shear stress reaches the shear strength of the masonry material. In particular, in the examined case, the shear strength of each structural unit along the two main analysis directions (x and y), chosen according to the load-bearing wall principal axes, has been calculated according to the following relationship, expressed for simplicity with reference to the direction x only:

$$F_{\mathrm{SLV},xi} = \frac{\mu_{xi}\xi_{xi}\zeta_x A_{xi}\tau_{\mathrm{d}i}}{\beta_{xi}k_i} \tag{1}$$

where:

- A_{xi} is the shear-resistant area of walls of the ith floor in the direction x, by considering also panels with inclination within $\pm 45°$ having an effective area reduced by the factor $\cos\alpha$;

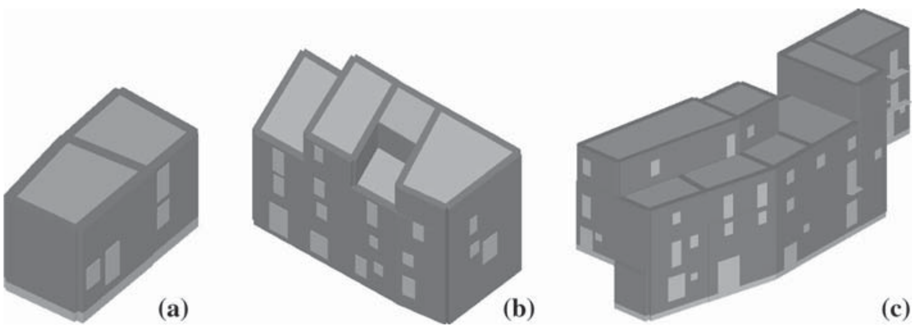


Fig. 15 3MURI macro-element models of the aggregates type A (**a**), type B (**b**) and type C (**c**)

- τ_{di} is the design value of the shear strength of masonry piers of the ith floor, calculated as follows:

$$\tau_{di} = \tau_{0d}\sqrt{1 + \frac{\sigma_{0i}}{1.5\tau_{0d}}} \tag{2}$$

  where τ_{0d} is the masonry design shear strength, assessed taking into account the confidence factor, and σ_{0i} is the medium vertical stress on the surface of the ith floor walls;
- k_i is the ratio between the resultant of ith floor seismic forces and the total seismic force;
- β_{xi} is a plan irregularity coefficient at the ith floor, given by the following expression:

$$\beta_{xi} = 1 + 2\frac{e_{yi}}{d_{yi}} \le 1.25 \tag{3}$$

  where e_{yi} is the eccentricity between barycentre and centre of stiffness and d_{yi} is the distance between the barycentre and the outer wall in the direction x;
- μ_{xi} is a coefficient considering the stiffness and strength masonry walls homogeneity, which can thus be assessed:

$$\mu_{xi} = 1 - 0.2\sqrt{\frac{N_{mxi}\sum_j A_{xi,j}^2}{A_{xi}^2}} \tag{4}$$

  where N_{mxi} and $A_{xi,j}$ are, respectively, the number of masonry walls and the generic masonry pier area in the direction x at the ith floor (the sum is extended to all masonry piers of the ith floor, so that $\Sigma_j A_{xi,j} = A_{xi}$);
- ξ_{xi} is a coefficient related to the failure type expected in masonry walls at the ith floor. It assumes value 1 in case of shear collapse, while it may be equal to 0.8 in case of compression-bending collapse (slender masonry piers, slightly vertically loaded or in the presence of weak spandrels);
- ζ_x is a coefficient related to the spandrel strength of masonry walls arranged in the direction x: it is equal to 1 for strong spandrels (collapse of vertical masonry piers), while it may assume a smaller value (up to 0.8) in the case of weak spandrels not able to block the rotation of masonry piers edges.

3.2.4 Presentation and comparison of results

The numerical analyses on the aggregate type A have provided the curves depicted in Fig. 16, where it is clearly shown that with both programs (3MURI and SAP2000), almost the same response in terms of strength and stiffness is achieved. About ductility, there is instead a substantial difference between programs, since SAP2000 is able to capture a strength reduction when plastic hinges exceed ultimate deformation and advance in the plastic field (branch with zero resistance), whereas with 3MURI, no more residual strength is provided by failed elements.

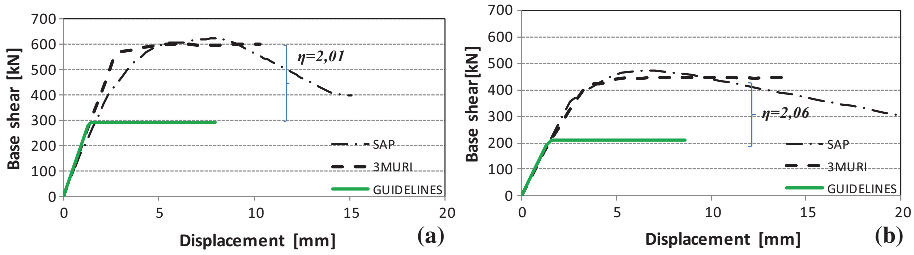


Fig. 16 Comparison among nonlinear responses of the aggregate type A in directions x (**a**) and y (**b**)

Determination of requested displacement is made assuming a structure elastic–perfectly plastic behaviour, i.e. by replacing real capacity curve with an equivalent bilateral curve from energetic point of view, that is with an equal subtended area.

3MURI automatically performs the transition from MDOF structure to the equivalent SDOF system and gives as output the checks based on the comparison between the structure displacement demand (different at dissimilar limit states) and the structure displacement capacity.

Subsequently, estimation of the nonlinear response of each case study has been faced in a simple way by considering the contributions of every structural units, which are summed aiming at providing the global aggregate response. First, hand calculation based on the displacement congruence between adjacent structural units has been performed to evaluate the structural unit yielding displacement and, consequently, its stiffness. After, code provisions have been applied to assess the ultimate displacement of each unit. Finally, the LV1 approach of the Italian Guidelines on Cultural Heritage has been applied to the case study in order to assess the strength of each structural unit.

In Fig. 16, the simplified pushover curves deriving from the above procedure for each analysis direction have been plotted and compared to the results of sophisticated numerical analyses. From comparison, it is apparent that the aggregate shear strengths in directions x and y deriving from application of Guidelines are, respectively, 2.01 and 2.06 times less than those obtained with the 3MURI software.

The same results have been also confirmed for aggregate type B (Fig. 17), where the aggregate shear strengths in directions x and y deriving from application of Guidelines are, respectively, 2.11 and 2.08 times less than those obtained with the 3MURI software.

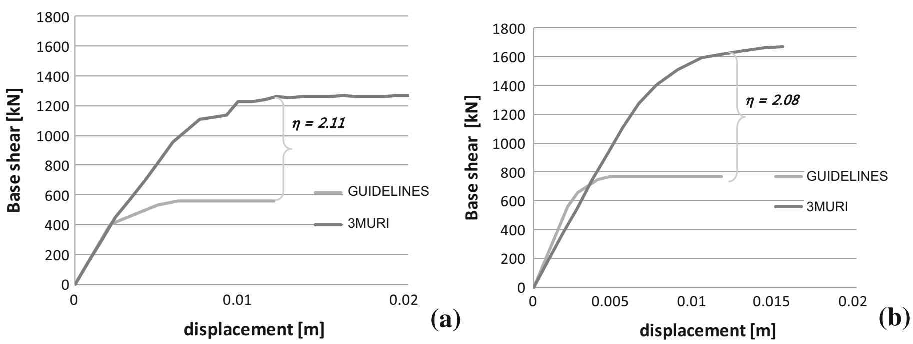


Fig. 17 Comparison among nonlinear responses of the aggregate type B in directions x (**a**) and y (**b**)

Identical situation is more or less for aggregate type C, where theoretical pushover curves have maximum shears in directions x and y about 2.05 and 2.00 times less than ones achieved from numerical analyses, respectively (Fig. 18).

It can be noticed that the detected differences can be mainly attributed to the different way used by Guidelines and Italian technical code to evaluate the shear stress of masonry walls.

In fact, for existing masonry buildings, both Italian Guidelines and Ministerial Circular no. 617 refer to in-plane shear strength of masonry panels measured according to a diagonal cracking failure criterion. Such a condition occurs when the main tensile stress in the panel centre reaches the masonry calculation resistance f_{td}. Nevertheless, the relationships used by Guidelines and Circular in calculating the ultimate shear stress value $\tau_{m,ult}$ are different. Both expressions are reported in the following with same symbols in order to allow for a useful comparison:

$$\tau_{m,\,ult} = \frac{1.5 f_{vd0}}{b}\sqrt{1 + \frac{\sigma_n}{1.5 f_{vd0}}} \tag{5}$$

$$\tau_{m,\,ult} = f_{vd0}\sqrt{1 + \frac{\sigma_n}{1.5 f_{vd0}}} \tag{6}$$

By comparing the two relationships, it is clear that the detected difference is linked to the factor b, that is the masonry pier slenderness (height to thickness) considered in the Italian technical code. In fact, if $b = 1$ (stocky panel), the $\tau_{m,ult}$ indicated by the Circular is 1.5 times larger than the Guidelines shear stress. Furthermore, by observing the shear strength calculation formula given in Guidelines, i.e. in the direction x of the building ith floor, it is noticed that the resistance is penalized from parameters β_{xi}, μ_{xi} and ξ_{xi}.

For the sake of example, if we consider a single-storey building with strong masonry spandrels, i.e. provided both with tie beams and strong architraves, the coefficients k_i and ζ_i assume unit values.

Therefore, a correction factor η, intended as the ratio between the maximum base shear obtained from 3MURI and the one achieved from Guidelines, can be used to predict in a more correct way the building base shear accomplished with Guidelines according to the Italian codes (M. D. 08 and M. C. 09), it being expressed as follows:

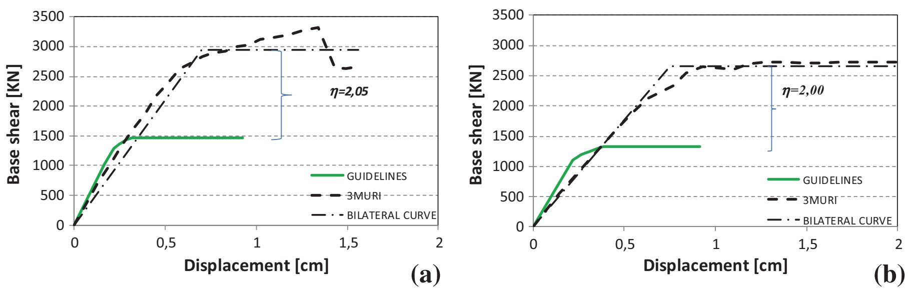


Fig. 18 Comparison among nonlinear responses of the aggregate type C in directions x (**a**) and y (**b**)

$$\eta = \frac{V^{\mathrm{MD08}}}{V^{\mathrm{LG}}} \tag{7}$$

For the sake of example, if we consider a mono-storey masonry building with strong spandrels, η takes the following relationship:

$$\eta = \frac{\beta}{\mu \cdot \xi} \cdot \frac{1.5}{b} \tag{8}$$

where b can assume values <1 (stocky piers) or >1.5 (slender piers).

As a result, by considering the possible variations of μ and ξ and the two limit values of b (1 and 1.5), the above concept can be generalized, providing ranges of values between 1.25 and 1.95 and 1.5 and 2.34 for slender and stocky piers, respectively. Moreover, by putting together the above results under graphical form, the design chart illustrated in Fig. 19 is provided. It can be usefully employed in order to know the correction factor η (dependent on the pier slenderness) to be used for correctly estimating the base shear of structural units into building aggregates starting from indications of Guidelines on Cultural Heritage.

Finally, for the aggregate type B the damage curves of the structural units, both isolated and inserted in the aggregate, have been derived for 4 different limit states, represented by the limit displacements $S_{d,1} = 0.70\ \delta_y$, $S_{d,\ 2} = 1.5\ \delta_y$, $S_{d,3} = 0.5(\delta_y + \delta_u)$ and $S_{d,4} = \delta_u$, where δ_y and δ_u are, respectively, the yielding displacement and the ultimate one of the building capacity curve (Lagomarsino and Giovinazzi 2006). In particular, the damage index is calculated, for each of the above limit states, as the ratio between the demand displacement (required by the earthquake) and the capacity one (achieved from the building capacity curve).

For the sake of example, the damage curves of S.U. 1 (heading unit) and S.U. 2 (intermediate unit) in direction y are given in Fig. 20, where the damage index μ_D,

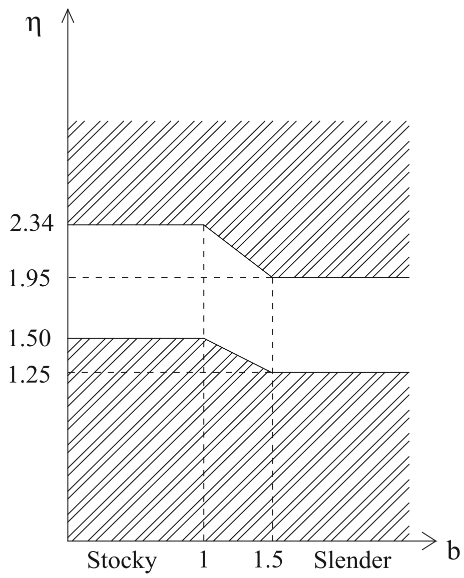


Fig. 19 Design chart for estimating the correction factor η

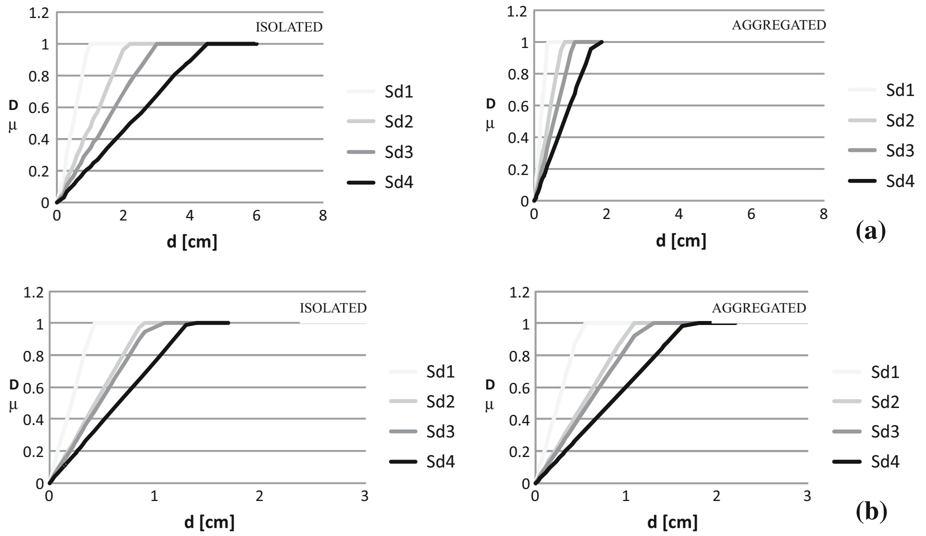


Fig. 20 Damage curves of the heading (**a**) and intermediate (**b**) S.U. of the aggregate type B

evaluated according to the EMS'98 scale (Grünthal 1998), is normalized within the range [0–1].

From these curves, it is apparent that for both structural units, the insertion into aggregates provides beneficial effects, since damage recorded for all limit state considered is reduced. This effect is more pronounced for the investigated heading unit. In fact, at the collapse limit state, the heading S.U. failure is attained at a displacement about 3.00 times greater than that of the same S.U. considered as isolated. On the other hand, for the intermediate aggregated S.U., collapse is achieved at a displacement about 1.30 times greater than the single S.U. one.

4 Global-scale analysis

After local analyses have been performed on the case studies, the damage analysis of a large built-up area of the village has been assessed. Firstly, in order to evaluate the damage indicator μ_D, according to the relationship defined in Cattari et al. (2004), a seismic micro-zoning of the territory has been faced. In particular, the geological characteristics of the area of *San Pio delle Camere* have been identified by means of the geological and micro-zoning maps of the Italian Civil Protection, where it is apparent that the inspected historic centre is located half on a stable area and the other half on a stable area susceptible to seismic amplification (Fig. 21) (Working Group MS-AQ 2010). However, it is worth to precise that the underground cavities, the so-called Grottoni or Camere, are not represented on the above-mentioned geological maps. In fact, most caves were detected during the post-earthquake survey activity performed by the Research Group of the University of Pisa coordinated by Mauro Sassu, which carried out some surveys propaedeutic for the

Fig. 21 Geologic map of *San Pio delle Camere* (Working Group MS-AQ 2010)

reconstruction plan of the municipality of *San Pio delle Camere* affected by the 2009 L'Aquila earthquake.

All collected data have been put in a GIS database, in order to apply the seismic vulnerability quick form illustrated in Table 1 and detailed in Formisano et al. (2010b, c).

This vulnerability assessment form has been adopted with some small adjustments by the Italian National Group Against Earthquakes (GNDT) as first screening tool for vulnerability assessment of masonry and r.c. buildings belonging to historical centres (Cherubini et al. 2000).

In order to consider the structural interaction among adjacent buildings, not considered in the cited method, a new form has been ideated by adding to the basic ten parameters of the original form new five parameters taking into account interaction effects among aggregate structural units under earthquakes. These factors, in part derived from previous studies found in the literature (Giovinazzi and Lagomarsino 2006), are:

1. In-elevation interaction;
2. Plan interaction;
3. Number of staggered floors;
4. Structural or typological heterogeneity among adjacent structural units;
5. Percentage difference in opening areas among adjacent facades.

Table 1 New vulnerability assessment form proposed for buildings grouped into aggregates

Parameter	Class score (*s*)				Weight (w)
	A	*B*	*C*	*D*	
1. Organization of vertical structures	0	5	20	45	1
2. Nature of vertical structures	0	5	25	45	0.25
3. Location of the building and type of foundation	0	5	25	45	0.75
4. Distribution of plan resisting elements	0	5	25	45	1.5
5. In-plane regularity	0	5	25	45	0.5
6. Vertical regularity	0	5	25	45	0.5−1
7. Type of floor	0	5	15	45	0.75−1
8. Roofing	0	15	25	45	0.75
9. Details	0	0	25	45	0.25
10. Physical conditions	0	5	25	45	1
11. Presence of adjacent buildings with different heights	−20	0	15	45	1
12. Position of the building in the aggregate	−45	−25	−15	0	1.5
13. Number of staggered floors	0	15	25	45	0.5
14. Structural or typological heterogeneity among adjacent structural units	−15	−10	0	45	1.2
15. Percentage difference in opening areas among adjacent facades	−20	0	25	45	1

The new parameter weights to be considered in the implemented quick survey form were determined according to previous studies (Formisano et al. 2011). Therefore, on the basis of the vulnerability indexes achieved from the survey form for all structural units, the forecast damage map of the historical centre of *San Pio delle Camere* has been developed according to an analysis method already applied to other Italian historical centres (Formisano et al. 2015).

The method has been applied to 128 masonry aggregates, composed by 413 structural units. Thus, 413 vulnerability indexes (I_V) have been calculated in order to estimate the mean damage grade μ_D for a value of the seismic intensity I equal to 10. The predicted damage map is depicted in Fig. 22.

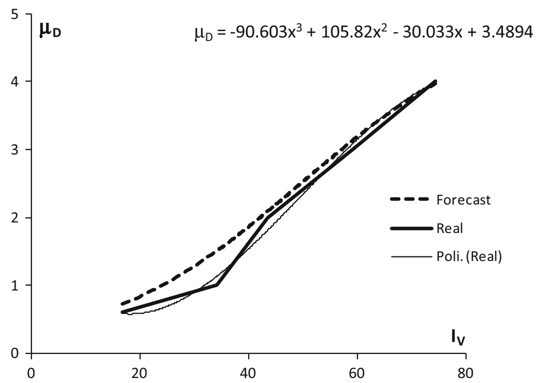
From this map, it is apparent that about 65% of the examined aggregates is characterized by heavy damages, 25% by moderate damages, 7% by very heavy damages, 1% by collapses and, finally, 2% without damages.

The vulnerability index I_V and the damage grade μ_D have been correlated with each other through a third-degree polynomial relationship, graphically depicted in Fig. 23. The diagram shows the comparison between calculated expected damage and effectively occurred one. In this case, it may be observed a good agreement between the curve of the predicted damage and that of the real damage. In fact, after values of the vulnerability index equal to 45, the two curves are almost coincident. Moreover, the forecast damages are greater than those occurred under the earthquake.

As a result, the numerical procedure for assessing the damage based on the seismic vulnerability index achieved from the proposed quick form is on the safe side in predicting

Fig. 22 Predicted damage map of *San Pio delle Camere*

Fig. 23 Comparison among forecast damages and real ones occurred in *San Pio delle Camere*



the seismic behaviour of investigated masonry building aggregates. Further studies on other building compounds belonging to other zones prone to earthquakes are needed to extend these results to the whole Italian territory.

5 Concluding remarks

In the early part of the paper, three masonry building compounds have been seismically investigated in the nonlinear static field through three different analysis approaches, namely the Italian Guidelines on Cultural Heritage, the 3MURI calculation program for masonry structures and the SAP2000 structural analysis program. The latter software, which was used to precisely assess the stiffness of deformable floors of one of three examined building aggregates, has provided pushover curves very similar to 3MURI ones, confirming the effectiveness of the equivalent frame technique to estimate the seismic response of masonry buildings.

Later on, based on the calculation programs results, a simple nonlinear methodology has been set up aiming at plotting simplified pushover curves of both single structural units and building compounds. The achieved results have shown that Italian Guidelines on Cultural Heritage furnish precautionary results, with aggregate base shears almost one-half of 3MURI ones. The different results obtained with this simplified analysis method have allowed to set up a chart, where the calculation program base shear-to-the Guidelines one ratio, indicated as correction factor η, has been appraised on the basis of the wall slenderness. As a result, the correction factor has been found to be variable between 1.25 and 1.95 for slender piers (slenderness of 1.5) and between 1.5 and 2.34 for stocky piers (slenderness of 1.0). Lastly, the beneficial aggregate effect on the seismic behaviour of structural units has been demonstrated. In fact, when inserted into aggregates, structural units have shown less damages than those recorded when they are considered as single buildings. This effect is more pronounced for heading units than intermediate ones.

Subsequently, in the second part of the paper, the validation of a seismic vulnerability and damage assessment procedure for masonry building aggregates of the examined historical centre has been done. Firstly, the evaluation of the predicted seismic damages of the aggregates has been done. Subsequently, the real damages have been visually identified according to the EMS'98 damage scale. Finally, the predicted damages, correlated with the building vulnerability indexes, have been compared with the real ones. From comparison, it has been declared that the proposed method is conservative and, therefore, effective, in predicting the damage suffered from building aggregates of *San Pio delle Camere* under earthquakes. However, further studies on other building compounds belonging to different Italian zones are needed to extend the obtained results and, therefore, to generalize the implemented analysis procedure.

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