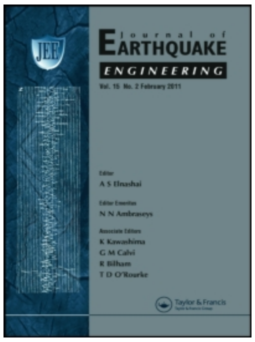

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Theoretical and Numerical Seismic Analysis of Masonry Building Aggregates: Case Studies in San Pio Delle Camere (L'Aquila, Italy)

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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 behavior. For this reason, in this article a simple methodology to forecast the masonry aggregate seismic response has been set up. 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.

Keywords Masonry Building Compounds; Static Nonlinear Analysis; Macro-Element Modeling; Italian Guidelines on Cultural Heritage; Design Chart; Damage Curves

1. Introduction

The historic center 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 behavior on the basis of unclear criteria. In particular, the case of building aggregates represents the norm within roughly all Italian towns [Giuffrè, 1993].

Aggregated buildings represent, in fact, an important and typical peculiarity in many Italian old town centers. Most common aggregated building type are continuous curtains of masonry buildings developed along a way with different total height, story 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 studied after recent seismic events affecting the Italian area. The L'Aquila earthquake and, more recently, the Emilia-Romagna one, demonstrated that aggregated buildings generally show a group behavior which improves seismic performances of the component structural units, also when they are made of low quality masonry [Formisano, 2012a; Formisano *et al.*, 2010a, 2010b, 2011, 2015; Indirli *et al.*, 2013].

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According to the recent relevant code prescriptions about 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 noting that an aggregate is composed by a group of not homogeneous structural units interacting each other during earthquakes. So, an aggregate is made by more 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” [M. C. 02/02/09 n. 617, 2009]. 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 behavior toward static and dynamic loads.

In the literature, different approaches have been presented for studying the behavior 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 Lourenco, 2004; Senaldi *et al*., 2010].

Moreover, interesting and relevant standard provisions used for a lot of 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 appraise 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 behavior (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 this article is to extend the indications of the above Guidelines for predicting the seismic global behavior in terms of shear strength of some historical building aggregates placed in San Pio delle Camere, a town in the district of L’Aquila (Italy). Although the occurrence of local mechanisms is very diffused into historical masonry building compounds and, therefore, deserve the attention of the scientific community, the presence of both some steel tie-beams and effective floor-to-wall connections avoided first mode collapse mechanisms into inspected buildings, so pushing the author to investigate their global behavior only.

Accordingly, 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 behavioral differences of the former when they are within building compounds.

The final study target, which represents the research future development, is to deepen the seismic behavior of historical building aggregates aiming at both achieving and comparing the fragility curves of both single units and aggregates ones. As a result, the beneficial or detrimental effect deriving from grouping in aggregate will be shown for heading, corner, and intermediate structural units belonging to clustered buildings.

2. The *San Pio delle Camere* Old Town center

San Pio delle Camere is a little town with medieval origin sited in the district of L’Aquila at the mountainside of the *Monte Gentile* along the Aterno valley (Fig. 1). The appellations “chambers” (“delle Camere” in Italian) or “caves” refer to the characteristic caves located

FIGURE 1 Landscape of *San Pio delle Camere.*

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 16th 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 medieval 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.

These medieval 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].

FIGURE 2 The S. Pio Church in the historical center of *San Pio delle Camere.*

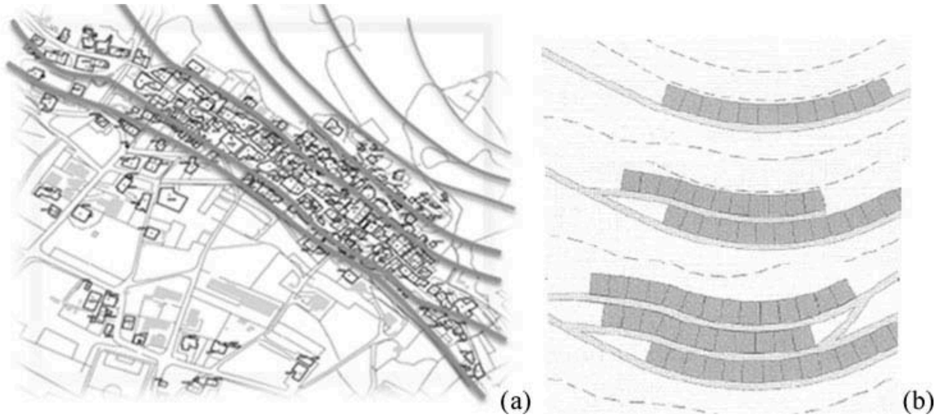

FIGURE 3 The *San Pio delle Camere* building map (a) and the building aggregation parallel to the contour lines (b).

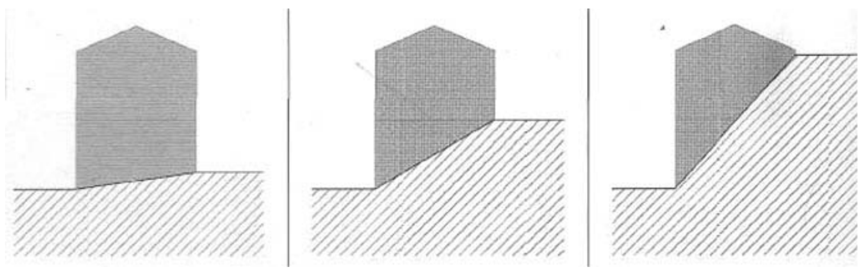

FIGURE 4 Positions of buildings as respect to the ground natural slope.

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 center, aggregated buildings depicted in Fig. 6 are placed mainly on staggered levels. Masonry texture is not regular but, in spite of this, it shows a good apparatus with some distinctive features, like medium-sized stones, horizontal layers, and small size wedges guarantying the contact among all elements (Fig. 7) [Formisano, 2012b]. Consequently, the continuity of masonry walls giving rise to unique blocks in the own plane is assured, but the absence of headers into their transversal section is noticed.

Buildings have three floors at most, whereas only in few cases they developed on four levels. Low homogeneity with original buildings parts is observed in raised volumes that are realized with either full or perforated bricks or concrete blocks. Single-layer brick, 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 behavior, represent the main roofing structure.

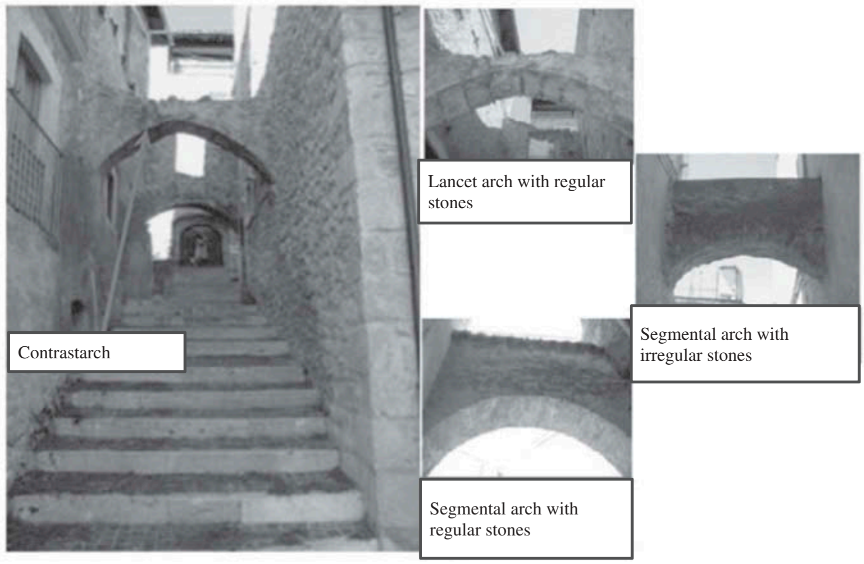

FIGURE 5 Contrast arches in the historic center.

FIGURE 6 Map of the historical building compounds.

3. The Case Studies

Aggregated buildings situated in the *San Pio delle Camere* old town center showed diffused structural damages due to the April 6, 2009 earthquake [Formisano *et al*., 2013].

FIGURE 7 Typical masonry textures.

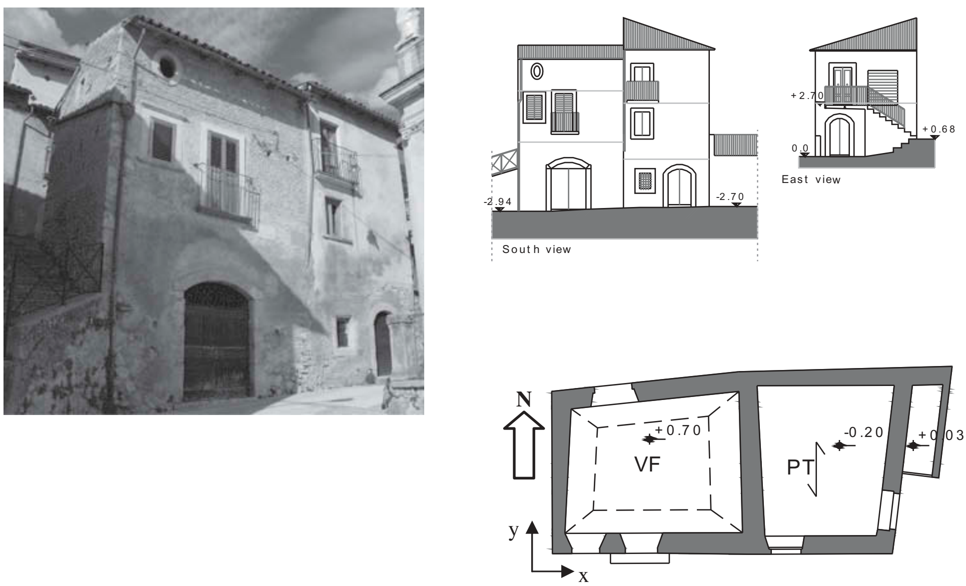

FIGURE 8 The aggregate type A: main view and architectural drawings.

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.

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 x 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 x 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 shape stones sustaining

FIGURE 9 Main views of the aggregate type B.

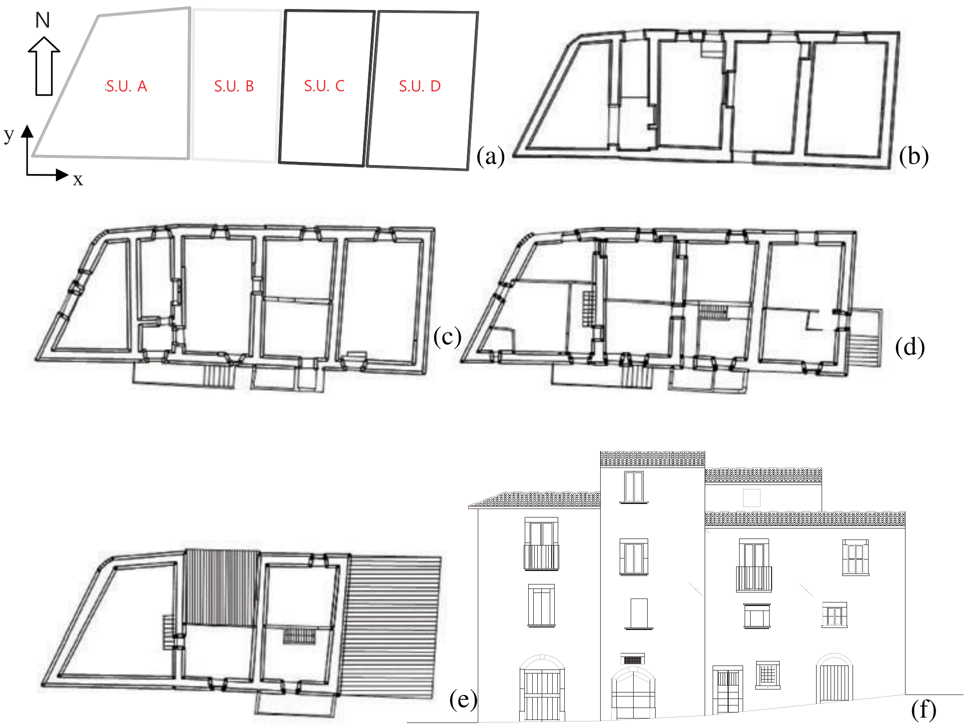

FIGURE 10 The aggregate type B: general plan layout (a), underground floor (b), ground floor (c), first floor (d), second floor (e), and north side view (f).

barrel and pavilion vaults. Building facades result to be aligned and staggered floors are missing.

Aggregate type C is based on a 65% inclined ground and can be inscribed into a 34.0 m x 14.0 m rectangular area. The aggregate structural units are made of a limestone masonry with rough and irregular stones, having poor quality transversal connections realized with bondstones 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. Some

FIGURE 11 The aggregate type C: main view and drawings.

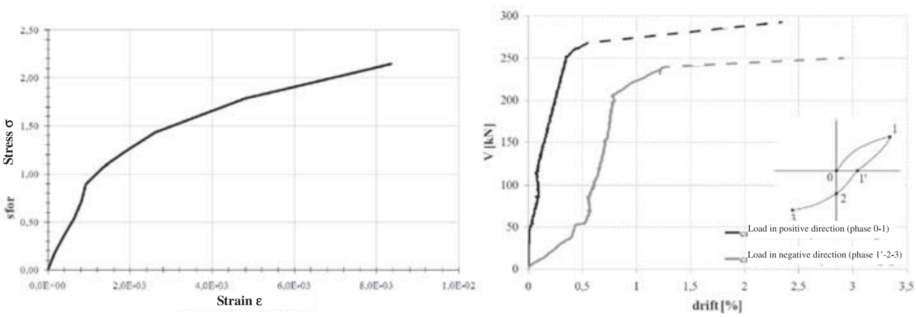

FIGURE 12 Experimental responses of Abruzzo masonry panels.

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 were 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]. Such tests have provided the following mechanical features: γ_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) (Fig. 12).

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 in situ checks on materials have been carried out and few architectural details have been investigated. On the other hand, Italian Circular n.617/09 [2009] specifies that γ_m is one when nonlinear static analyses are performed.

4. Numerical Modeling and Theoretical Analysis

4.1. Foreword

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

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, difficult to be evaluated when flexible horizontal structures are of concern. Afterward, 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 these numerical analyses have conducted, as shown in Sec. 5, toward a simple indication how to better predict, from the theoretical point of view, the 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* [MiBAC, 2011].

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.

4.2. 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 modeling is used for masonry buildings, vertical beam elements representing masonry walls into two principal directions cannot be aligned (Fig. 13).

An important modeling 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 modeling with two diagonal trusses arranged according to a St. Andrew’s cross configuration.

In the current study lumped plasticity modeling has been implemented for the equivalent frame members. In this way the progressive greater deformability connected to the plastic behavior extension is not considered, but the material nonlinearity connected 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

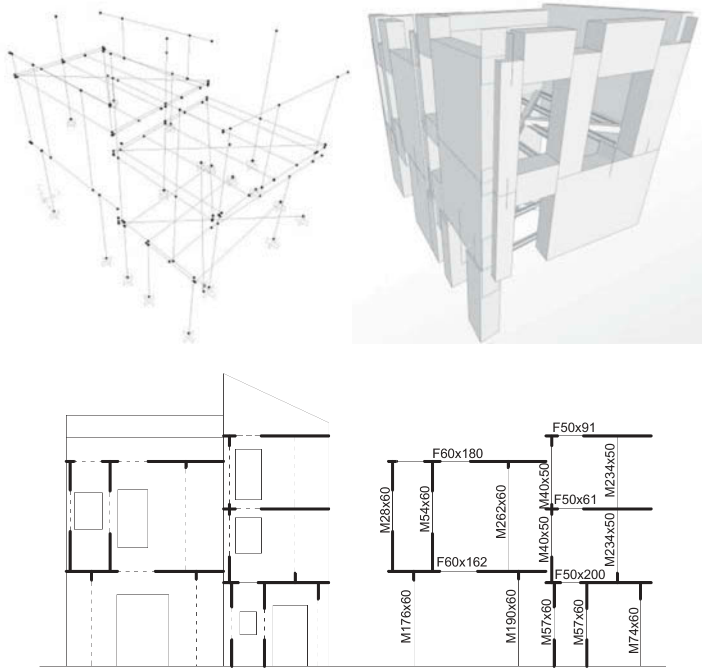

FIGURE 13 The equivalent frame model of the aggregate type A.

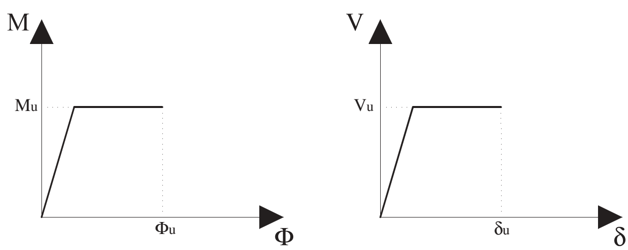

FIGURE 14 Plastic hinge laws for masonry structural elements.

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); and
- $\delta_u = 0.004$ for masonry piers and spandrels subjected to shear.

4.3. 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 modeling 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.

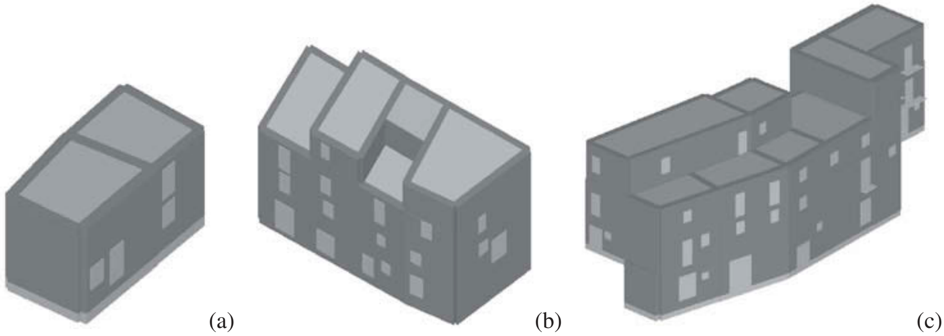

FIGURE 15 3Muri macro-element models of the aggregates type A (a), type B (b), and type C (c).

The program gives to the walls the role of resistant elements toward 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. The used modeling approach neglects the contribution of walls having own plane perpendicular to the seismic load direction due to their considerable flexibility (Fig. 15). Therefore, only walls placed along an assigned direction can be considered as seismic-resistant elements when seismic loads are applied along that direction.

4.4. Application of Italian Guidelines on Cultural Heritage

After the 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 behavior [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:

$$F_{SLV,xi} = \frac{\mu_{xi}\xi_{xi}\zeta_x A_{xi}\tau_{di}}{\beta_{xi}k_i}. \tag{1}$$

where

- A_{xi} is the shear resistant area of walls of the *i-th* floor in the direction x, by considering also panels with inclination within $\pm$ 45° having an effective area reduced by the factor $cos\alpha$;
- τ_{di} is the design value of the shear strength of masonry piers of the *i-th* 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 *i-th* floor walls;

- k_i is the ratio between the resultant of *i-th* floor seismic forces and the total seismic force;
- β_{xi} is a plan irregularity coefficient at the *i-th* floor, given by the following expression:

$$\beta_{xi} = 1 + 2\frac{e_{yi}}{d_{yi}} \leq 1,25, \tag{3}$$

where e_{yi} is the eccentricity between barycenter and center of stiffness and d_{yi} is the distance between the barycenter 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:

$$\mu_{xi} = 1 - 0,2\sqrt{\frac{N_{mxi}\sum_j A^2_{xi,j}}{A^2_{xi}}}, \tag{4}$$

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 *i–th* floor (the sum is extended to all masonry piers of the *i–th* floor $\Sigma_j A_{xi,j}=A_{xi}$);

- ξ_{xi} is a coefficient related to the failure type expected in masonry walls at the *i-th* 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); and
- 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.

5. 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 pushover curve 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.

Determination of requested displacement is made assuming a structure elastic-perfectly plastic behavior, 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, provisions based on the Italian

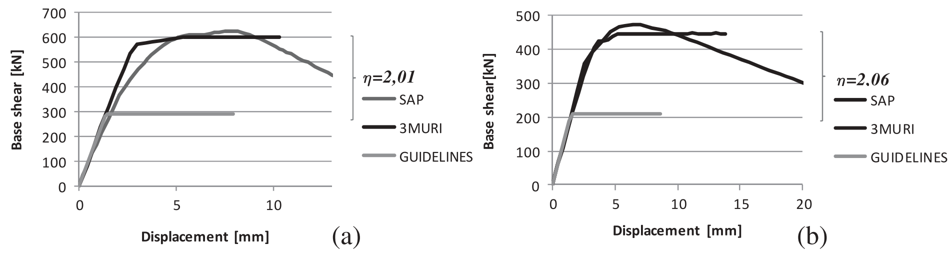

FIGURE 16 Comparison among nonlinear responses of the aggregate type A in direction *x* (a) and *y* (b).

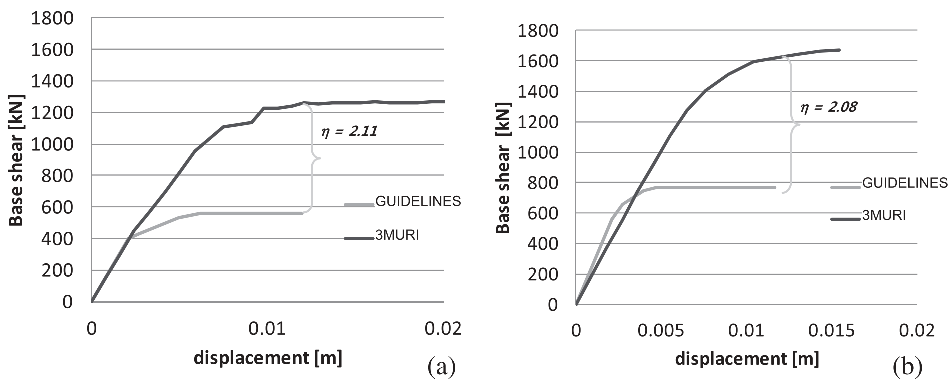

FIGURE 17 Comparison among nonlinear responses of the aggregate type B in direction *x* (a) and *y* (b).

ReLUIS guidelines [2010] have been used to evaluate the yielding displacement of S.U. Subsequently, the same 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 their in-plane strength.

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 direction *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 direction *x* and *y* deriving from application of Guidelines are, respectively, 2.11 and 2.08 times less than those obtained with 3MURI software. 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 codes [M.D. 14/01/08, 2008; M.C. 02/02/09 n. 617, 2009] to evaluate the shear stress of masonry walls.

In fact, for existing masonry buildings, both Italian Guidelines and Ministerial Circular n. 617 [2009] 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

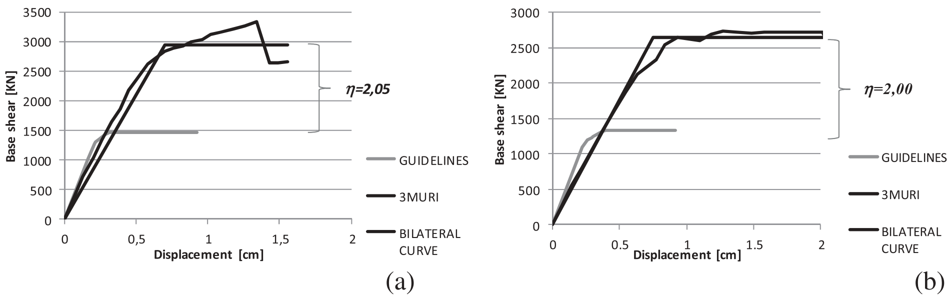

FIGURE 18 Comparison among nonlinear responses of the aggregate type C in direction *x* (a) and *y* (b).

in the panel center reaches the masonry calculation strength f_{td}. Nevertheless, the relationships used by Ministerial Circular (Eq. (5)) and Guidelines (Eq. (6)) 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 two relationships, it is clear that the detected difference is linked to the factor b, that is the masonry pier slenderness (height/thickness) considered in the Italian technical code. In fact if $b = 1$ (stocky panel), the $\tau_{m,ult}$ indicated by the Ministerial 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 i-th floor (see Eq. (1)), it is noticed that the strength $F_{SLV,xi}$ achieved from the evaluation of $\tau_{m,ult}$ (Eq. (6)) is penalized from parameters β_{xi}, μ_{xi} and ξ_{xi} with respect to the Ministerial Circular relationship (Eq. (5)). Nevertheless, if the building is sufficiently regular, these parameters should not be too punishing for the shear strength assessment. 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, based on provisions of the Italian Ministerial Decree of Public Works (MD 08), and the one achieved from Guidelines on Cultural Heritage (GCH), can be used to predict in a more correct way the building base shear as follows:

$$\eta = \frac{V^{MD08}}{V^{GCH}}. \tag{7}$$

For the sake of example, if we consider a mono-storey masonry building with strong spandrels, considering that $V^{MD08} = \tau_{m,ult}$ x A_i, η takes the following relationship:

$$\eta = \frac{\beta}{\mu \cdot \xi} \cdot \frac{1,5}{b}, \tag{8}$$

where b can assume values equal to 1 (stocky piers) or to 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 generalised, providing ranges of values between 1.25 and 1.95 and 1.5 and 2.34 for slender and stocky piers, respectively. The lower and upper values of these ranges have been obtained for both piers by considering the maximum and the minimum values of the above parameters, 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. In the case under study, for each analysis direction, η has been evaluated by assuming the pertinent value of b (1 or 1.5) related to the major number of masonry piers (stocky or slender, respectively) detected into every walls.

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]. The generic curve is defined as follows: after the capacity curve of the building is built, for different earthquake demand spectra, the resulting demand displacements are assessed and then compared to the building capacity one. The ratio between each demand displacement and the capacity one provides a damage index μ_D. This index can be framed within five ranges ([0÷0.2], [0.2÷0.4], [0.4÷0.6], [0.6÷0.8], [0.8÷1.0]) representing respectively the damage levels (slight, moderate, substantial-heavy, very heavy, destruction) reported in the EMS'98 scale, but normalised in the range [0÷1] [Grunthal, 1998]. In particular, when the ratio between the demand displacement and the capacity one is greater than one, the damage index always assumes a unitary value. Finally, the joining of the different points achieved by correlating each displacement demand to the corresponding damage index supplies the damage curve.

For the sake of example, the damage curves of S.U. 1 (heading unit) and S.U. 2 (intermediate unit) in direction y have been reported in Fig. 20. From these curves it is apparent that for both structural units, the insertion into aggregates provides beneficial effects, since damage recorded for all limit states considered is reduced. This effect is more pronounced

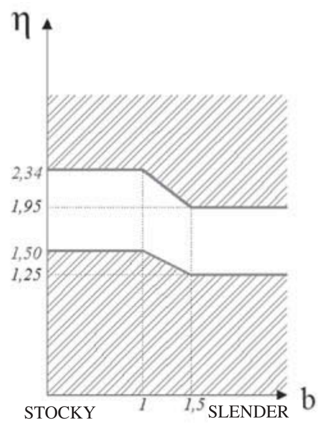

FIGURE 19 Design chart for estimating the correction factor η.

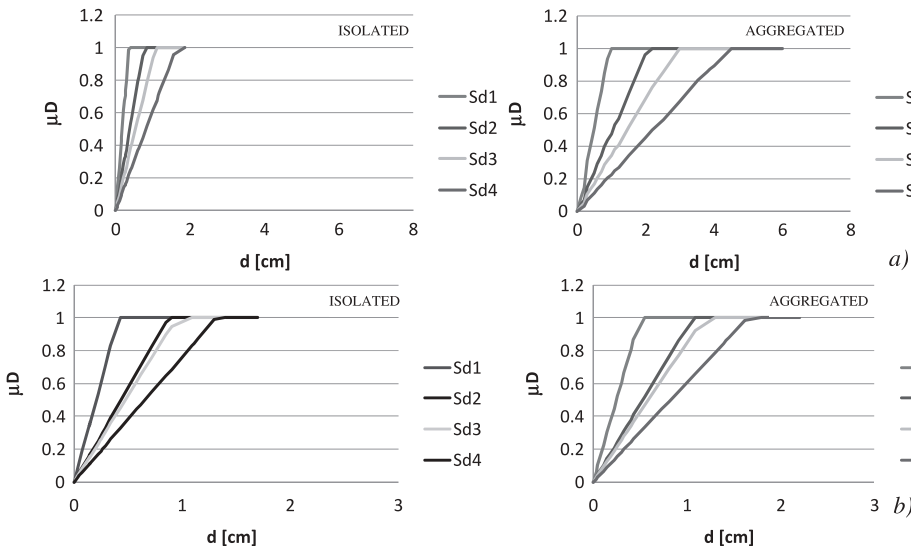

FIGURE 20 Damage curves of the heading (a) and intermediate (b) S.U. of the aggregate type B.

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.

6. Concluding Remarks

In this paper three masonry building compounds have been seismically investigated in the nonlinear static field through three different 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 on the global response of one of three examined building aggregates, has provided pushover curves very similar to 3MURI ones, confirming the effectiveness of a more general structural program also to estimate the seismic response of masonry buildings.

Subsequently, based on sophisticated numerical analysis results, a simple nonlinear methodology has been setup 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 furnishes precautionary results, with aggregate base shears almost one half of 3MURI ones. The different results obtained with Guidelines has allowed to setup a chart, where the base shear amplification factor with respect to that of the calculation program has been appraised on the basis of the type of masonry piers considered. In particular, this correction factor to correctly estimate the code base shear of structural units within building aggregates has been found to be variable between 1.25 and 1.95 and between 1.5 and 2.34 when slender (slenderness of 1.5) and stocky (slenderness of 1.0) piers are predominantly located in those buildings, respectively.

Finally, the beneficial aggregate effect on the seismic behavior of structural units has been demonstrated. In fact, when inserted into aggregates, structural units can sustain seismic displacement demands greater than those resisted by isolated buildings. This effect is more pronounced for the heading structural unit than the intermediate one, since the former are able to attain, at the collapse limit state, an ultimate displacement about three times greater than that of the isolated S.U., whereas the latter shows an ultimate displacement 1.30 times greater than the isolated S.U. one. As a result, the structural units belonging to the examined building compound suffer less seismic damages than those they could undergo when considered as single structures.

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